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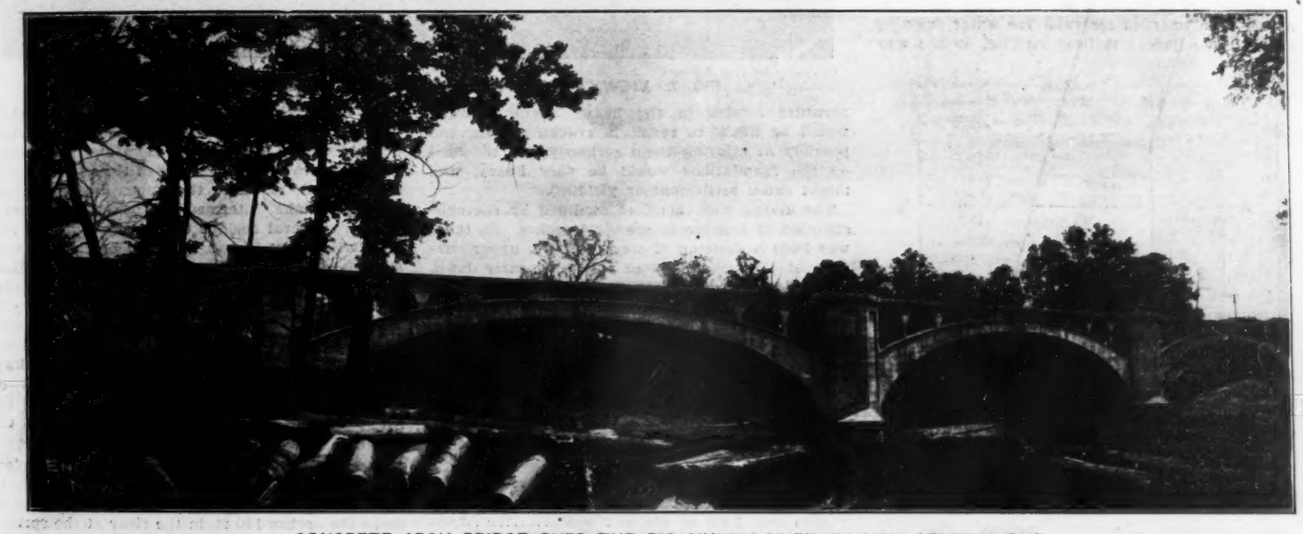
CONCRETE BRIDGE OVER THE BIG MUDDY RIVER; ILLINOIS CENTRAL R. R.

By H. W. Parkhurst, M. Am. Soc. C. E.*

At a distance of 304 miles from Chicago, the main line of the Illinois Central R. R. crosses the Big Muddy River, a stream draining a large section of southern Illinois, and discharging into the Mississippi River a few miles below Grand Tower. The railway company in double-tracking this portion of its line has built a concrete structure to take the place of the three-span, single-track Pratt truss bridge, which had been built about 1889, and which in 1901 was already getting too light for the heavy traffic. The new bridge, which is now in regular use, has three elliptical arches, each of 140 ft. clear span and 30 ft. rise above the springing lines. Its construction was begun in the autumn of 1901, and it was finished in readiness for use on May 10, 1903, thus requiring about 20 months for the work. During all this time, however, the regular traffic of the road was maintained with practically no delays or interference, one track always being kept open.



CONCRETE ARCH BRIDGE OVER THE BIG MUDDY RIVER, ILLINOIS CENTRAL R. R.
H. W. Parkhurst, M. Am. Soc. C. E., Engineer of Bridges, Illinois Central R. R. G. H. Scribner, Jr., Chicago, Ill., Contractor.

FOUNDATIONS.

The structure in service when the new work was begun was a single-track bridge, of three through Pratt truss spans, resting on masonry piers and abutments. Each span was 161 ft. c. to c. of piers. The masonry had pile foundations, which, as well as the footing courses of the piers, were put in for the full width needed for double-track spans. The upper parts of the piers, however, were only built long enough to carry the single-track structure, the ends being toothed for bonding in the additions when they should be built.

Borings were made at each pier and abutment to determine the character of the underlying material. Rock was found to be at a great depth below the base of the rail, and this was overlaid with horizontal strata of shale which gradually changed to a tough and then to a more plastic clay at about the elevation of the bottom of the old masonry. The latter was generally in sound condition, though the north abutment had cracked and settled a little, and it was apparently unwise to attempt to go to rock for the foundations of the new structure, since the old masonry, if built into and surrounded by the new, would assist materially in making up the requisite mass for supporting the large arches (Fig. 1). It was, therefore, determined to drive additional piles around the old foundations, but also to place the base of the added masonry at a much lower level than that of the old, this difference being from 4 to 13 ft., depending upon conditions, which varied in each case.

Piles for these added foundations were generally of oak, and were driven by a creeping driver around the two piers, and by derrick and creeping drivers around the abutments. The creeping drivers were generally placed on the ground, and moved about the piers or abutments, and used piles 30 to 35 ft. long. Derrick drivers carrying a set of leads swung from a boom were used generally to drive the falsework piles, and also for some of the foundation piles. They were of a style designed and built by Mr. O. J. Travis, then Superintendent of Bridges, and were very efficient, it being possible to drive piles in cramped and awkward places where ordinary track drivers could not be used. These derrick drivers worked sometimes from the old bridge, and sometimes from the top of the falsework, which was built out from both sides towards the center.

It is probable that the earlier piers were built of sandstone, and that just prior to the erection of the last iron bridge, they had been rebuilt, the stone from the old work having been dumped promiscuously around the base of the new. These pieces of old dimension stone were seriously in the way, and had to be excavated around, grappled and hoisted out, to do which generally required cofferdam construction and excavation in advance of or coincident with the pile driving. Cofferdams were generally constructed of a single row of piles spaced about 3 ft. c. to c. Waling timbers from 6 × 12 ins. to 12 × 12 ins. were bolted to the piles, and a single row of 3-in. sheeting was generally sufficient in the tough clay to make a water-tight dam. These were kept dry by diaphragm pumps or buckets, or by a small centrifugal pump, as was found necessary. Fortunately the river kept at a low stage, and gave but little trouble during the foundation work. At ordinary low water the river is only 60 to 80 ft. wide, and runs through the center span, neither pier being

*Engineer of Bridges and Buildings, Illinois Central R. R., Chicago.

in the river, while the north abutment is rarely and the south abutment is almost never in the water. These conditions made the foundation work comparatively easy of access, and the work on it was prosecuted continuously during the first winter, and both piers were finished up to the spring line of the arches on Jan. 10, 1902.

The north abutment of the old structure had for some years shown a bad vertical crack where the tee end joined the main pier, and the latter had settled and moved a little toward the river. The reason for this developed during the progress of the foundation work around this abutment. The pile foundation under the old masonry had been cut off at too high a level to be kept saturated, and the outer grillage timber (a flattened oak stick 18 × 12 ins.) and the tops of a few of the outer piles were found to be decayed. To make this good, the rotten material was removed, new piles were driven in front, and the clay was cut out, a small section at a time, from under the old footing, and replaced by concrete thoroughly rammed under the masonry. Although this work was not done without some additional settlement of the old masonry (8 to 10 ins.), the new concrete was so well put in, and was furthermore so bonded together by numerous pieces of scrap rail, that after getting the whole foundation complete up to the level of the old footing course, there was no further settlement, and there has since been no indication that this portion of the foundation is less firm than the rest.

The south abutment was founded on the abrupt slope of the south bank of the stream, at a much higher elevation than the other. It might, therefore, have shown similar decay in its piles and grillage. It was found, however, to be perfectly sound. This was probably due to the fact that the cofferdam in which it had been built served as a sort of submerged reservoir for water draining out of the adjacent railway cutting, on the way

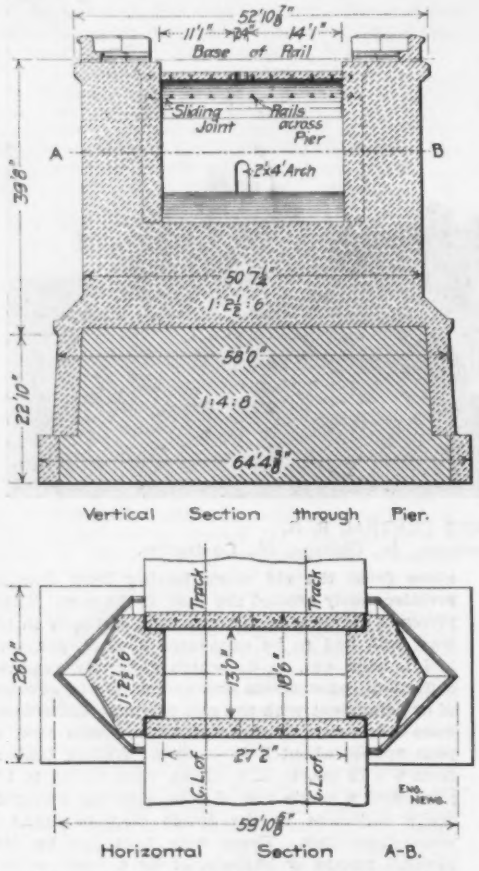


Fig. 1. Reconstructed Pier for Concrete Arch Bridge.

to the river. The foundation for the new work was carried down to a much lower level; a large portion of the clay, earth, etc., under the old foundation grillage was removed, and was replaced by concrete, this being done simultaneously with the work on the new portion of the foundation.

LOADS ON FOUNDATION.

The original design of the structure provided for earth filling over the spandrels, this to be held between heavy retaining walls, and to be covered with a suitable depth of ballast on which ordinary track ties should be placed. Serious objections to this plan were as follows: (1) The temperature strains in the heavy spandrel walls would be likely to result in cracking them and possibly in injuring them seriously; (2) the load on the foundations would be very heavy, and might cause settlement or yielding.

The design was therefore modified by the substitution of transverse spandrel arches. In these was built a skeleton of steel, and the upper surface of the concrete was finished nearly level, and about 2 ft. below the base of rail. This was covered with stone ballast (Fig. 2), ordinary hewed ties being bedded therein to carry the rails. Following is a comparison of loads on one pier under the original and revised plans:

Original plan:	Tons.
Main arch, 1,396 cu. yds. at 2 tons	2,792
Parapet and coping, 404 cu. yds. at 2 tons	808
Pier complete, including old masonry, 1,825 cu. yds. at 2 tons	3,650
Filling and ballast	1,600
Total	8,850

Revised plan:	Tons.
Main arch, 1,400 cu. yds. at 2 tons	2,800
Spandrel arches and copings, 600 cu. yds. at 2 tons	1,200
Pier complete, 1,555 cu. yds. at 2 tons	3,110
Ballast	525
Surplus weight of steel	25
Total	7,660

This shows a saving of nearly 1,200 tons on each pier foundation, or nearly 6 tons per pile.

It will be noted that the amount of concrete in the spandrel arches and copings in the revised plan is about 200 yds. greater than the corresponding material shown in the original plan. On the other hand, the pier in the new plan is considerably less heavy than in the old one. This saving of weight in the pier was made by constructing an arch within it, as shown, and explained further in connection with the subject of expansion. The principal saving was made by the omission of the earth filling.

GENERAL DESIGN.

It is possibly open to criticism that a bridge of three 140-ft. arches should have been used for the location, since the low-water width of the stream is very small, and adjacent banks are heavily timbered, and are apparently being gradually raised by deposits at high water from the stream itself. The reasons for using so long a bridge are as follows:

1. The fact that the old structure had three spans, each about 160 ft. long, made it necessary that ample waterway should be provided rather than incur the risk of suits for heavy damages at each subsequent period of high water.

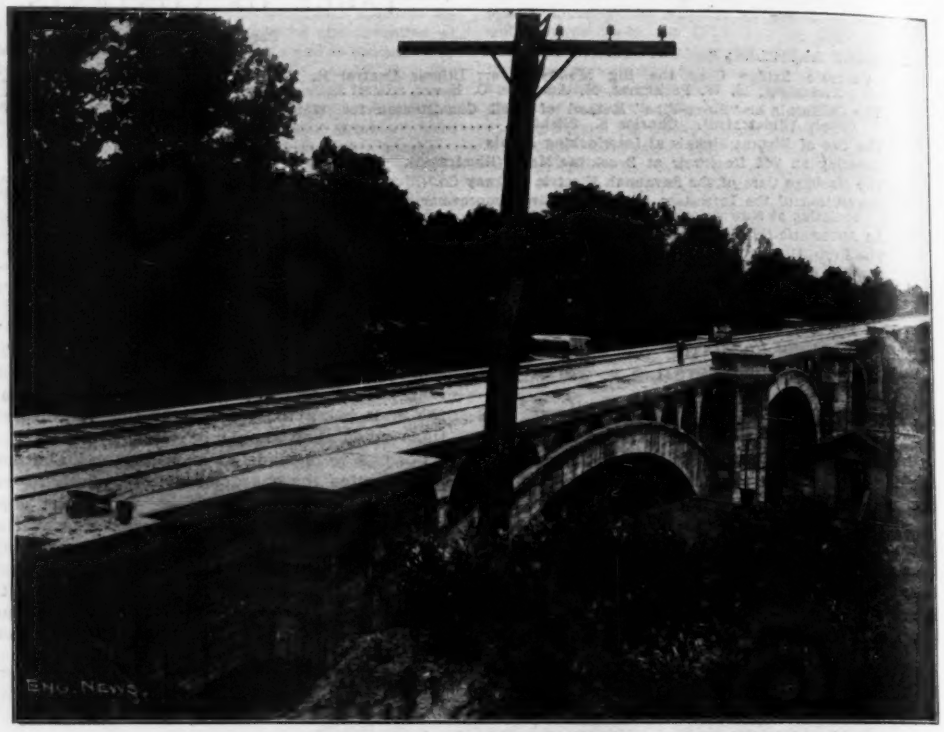


FIG. 2. VIEW OF CONCRETE ARCH, SHOWING BALLAST ROADBED.

2. The length of the waterway being thus determined, it was found to be the most economical plan to utilize the existing piers and abutments, as so much mass in the center of the new supports, and cheaper to make the longer arches, rather than to make intermediate piers and shorter arches. Several designs were worked out for this comparison, but it was found to be cheaper construction to reduce the number of intermediate piers to the minimum. Even with a rock foundation it will generally be found less expensive to make long spans, and few piers, providing sufficient rise can be had. In this case an additional rise of about 5 ft. was easily provided in taking out a sag in the general grade of the track at the crossing of the stream, thereby also reducing the grade at this point to the ruling grade on the division.

The distance from center to center of piers being about 161 ft. and the old piers being 9 ft. to 10 ft. thick at the base, it was finally decided to make the arches 140 ft. in the clear at the springing line, and the piers 21.5 ft. thick at that elevation, thus permitting the old abutments to be covered by 6 to 7 ft. of concrete. The height from the low-water level to the original base of rail was about 45 ft., and the rise of the arch was fixed at 30 ft., and the elevation of the springing line was so determined as to give about 2 ft. of ballast upon the upper concrete work. The span and rise of the arch being thus fixed by the natural conditions, the depth of the arch rib at the crown and at intermediate points from the crown to the piers was next to be determined. On this subject the writer quotes the following from Mr. J. M. Rankine: "The best course in practice is to assume a depth for the keystone according to an empirical rule, founded on dimensions of good existing examples of bridges." Rankine's formulas based on such examples are as follows:

1. Depth of keystone for a single arch, in feet, $= \sqrt{0.12}$ radius at crown.

2. Depth of keystone for an arch of a series, in feet, $= \sqrt{0.17}$ radius at crown.

These formulas depend on the radius at the crown; hence the determination of the form of the intrados is essential to the calculation of the

thickness of the crown. For a large arch, the choice lies between three forms: a segment of a circle, a basket-handled arch (or one formed with from three to five centers), and an ellipse. The segmental arch is objectionable on account of the difficulty of treatment of the connection of arch to pier or abutment, and on account of the apparent side thrust and tendency to overturn the intermediate piers.

The basket-handle or five-centered arch is usually an approximation to an ellipse, hence the true ellipse was selected as the proper line for the intrados. An ellipse of 140 ft. major and 60 ft. minor axes has a radius of curvature at its flattest point (the crown of the arch) of 167 ft. Applying the formula for a series of arches, we find that the crown should have a thickness of 5.33 ft. Inasmuch, however, as the arches at the center are heavily reinforced by the copings, as well as by the continuation of the spandrel work over the crown, the main rib of the arch was designed

1. Concrete is a brittle material, liable to crack irregularly, without defined stratification, under heavy shocks (such as derailment of trains or engines), and should therefore be tied together to prevent ultimate dismemberment. 2. The material with which this is to be done must be of such a nature that it can be protected from damage of any kind with the greatest possible care.

With these considerations, the placing of the steel skeleton in the center of the concrete naturally followed, rather than attempting to utilize its tensile strength by placing it near the outer lines and surfaces of the members of the structure. 3. The reinforcing material should be cheap, and should be designed with simple connections, inexpensive in construction, and easy to erect.

The writer doubts whether the almost universal effort which is being made to place the metal near the outside of composite structures is altogether desirable, and he would carefully avoid the use of thin metal near the outer surfaces, considerable importance, to provide for bond between the steel and the concrete, the mortar of the latter entering and extending through these holes and so firmly binding both together.

The steel structure was erected, adjusted and tightly bolted together by ordinary labor, and required no riveting and no expert assistants. Careful records show that large portions of it were completed at slightly less than ¼-ct. per lb. The self-supporting character of this reinforcing material was advantageous in several ways: (1) A level floor could easily be laid on the top, furnishing a very satisfactory working platform for making concrete. (2) Narrow runways were laid on it at trifling expense and used with confidence by the workmen. (3) The most important advantage was that the molds for the concrete could be wired to the steelwork, permitting cheaper construction in a part of the work where the quantity of concrete was small in proportion to amount of lumber needed for molds.

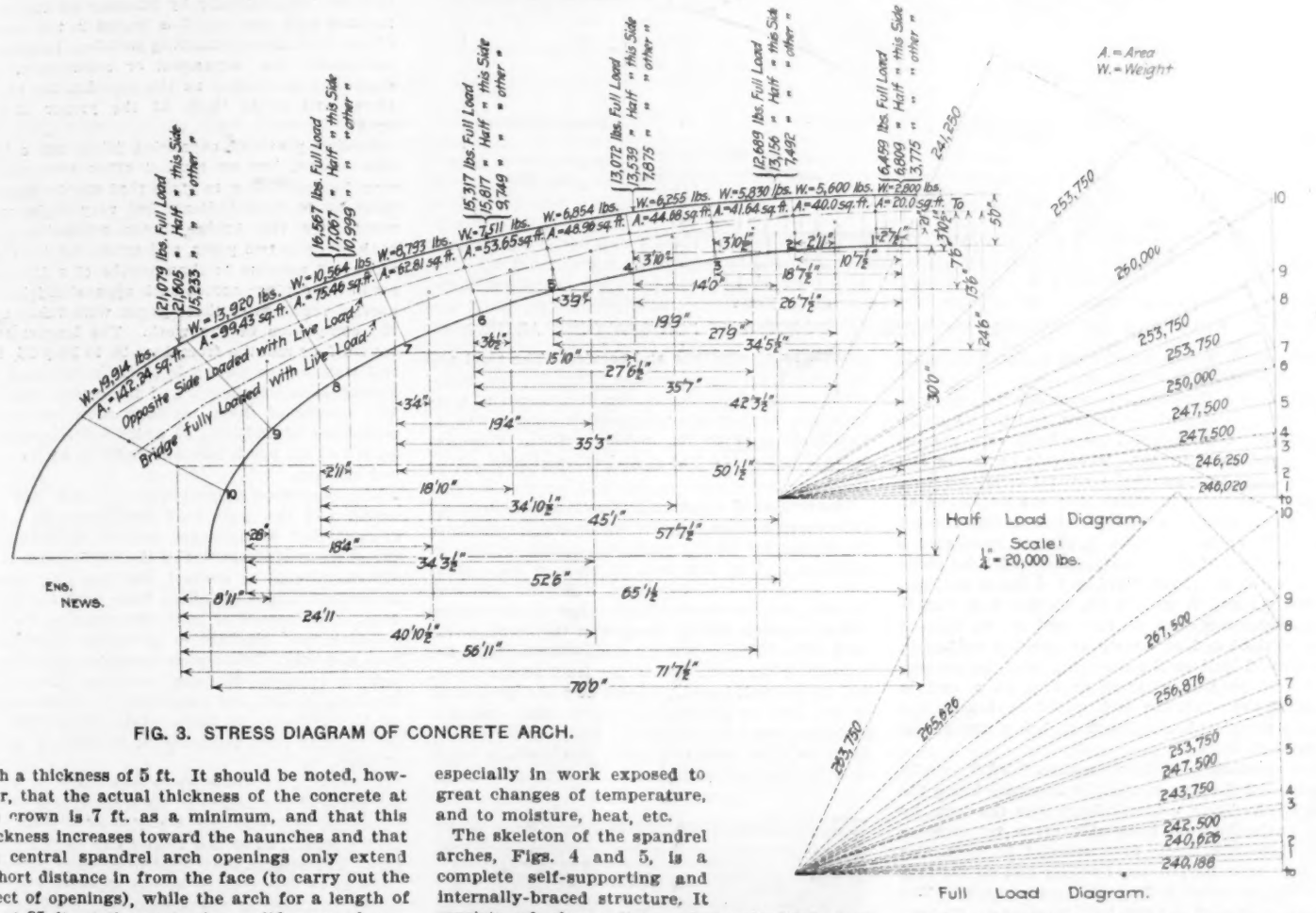


FIG. 3. STRESS DIAGRAM OF CONCRETE ARCH.

with a thickness of 5 ft. It should be noted, however, that the actual thickness of the concrete at the crown is 7 ft. as a minimum, and that this thickness increases toward the haunches and that the central spandrel arch openings only extend a short distance in from the face (to carry out the effect of openings), while the arch for a length of about 35 ft. at the center is a solid mass of concrete, with a minimum thickness of 7 ft.

The thickness at the haunches is determined by the ellipse of the extrados, which has axes of 180 and 70 ft., the major axis equalling the span of arch plus thickness of adjacent piers (nearly), and the minor axis adding twice the thickness of the arch crown to the axis of the ellipse of the intrados. Strain diagrams of full and partial loading were worked out, both for the original and modified designs. The diagram for the modified design is illustrated in Fig. 3.

The writer considered fully the construction of a steel skeleton reinforced with concrete for the main arch ribs, but could find no economy in the plan, and it was open to grave objections in the erection and construction work, besides being liable to cause delay for the manufacture and delivery of the steel. General plans and estimates were prepared, but were laid aside for the reasons alluded to.

As previously stated, the spandrel arch work was decided on mainly to reduce the load on foundations. It was slightly more expensive than the original plan of heavy walls and earth fillings. It was constructed on the following theory:

especially in work exposed to great changes of temperature, and to moisture, heat, etc.

The skeleton of the spandrel arches, Figs. 4 and 5, is a complete self-supporting and internally-braced structure. It consists of eleven lines of longitudinal horizontal steel rails, spaced 3 ft. c. to c., tied and also braced at this fixed distance by rods of 1-in. diameter, spaced about 18 ins. c. to c., each having two nuts at each end which are screwed up tightly against the webs of the rails. These webs are punched with 1⅛-in. holes at intervals of 3 ins. for their whole length, and any rod will fit and can be secured in any hole. The rails are joined end to end by the ordinary splice bars. This set of rails is held at proper elevation by vertical posts spaced also 3 ft. apart transversely, and at proper intervals for the spans of the arches. Each post is connected to the next by similar 1-in. rods, and is also connected to the longitudinal rail above it by adjustable diagonals which lie within the haunches of the arches. The vertical posts rest upon and are bolted to transverse horizontal rails set in recesses in the upper surface of the main rib of the concrete work. All rails have 1⅛-in. holes punched through their webs. This was done with a threefold purpose: (1) To save accurate templet work in laying out location of holes for connections; (2) to make it possible to add other connecting ties or struts or to make changes; and (3), in the writer's opinion, of

It may be of interest to consider the loads on the spandrel arches. The largest arch has a span of 15 ft.:

	Lbs.
Concrete in arch and copings, 60 cu. yds., at 2 tons, 120 tons	240,000
Steel skeleton	30,000
Ballast and track	130,000
Total fixed load	400,000

Assume moving load of three pair of drivers of a consolidation engine on each track with 50,000 lbs. per axle, making live load 300,000 lbs., or 700,000 lbs. of maximum combined load. To carry this there are 11 rails, 60 lbs. per yard, or 66 sq. ins. of steel and 66 sq. ft. section of concrete. The load would easily be carried by either, without undue strain, if flexure is not considered. The combination will unquestionably withstand any bending moments that may arise from partial or unequal loading.

EXPANSION JOINTS.

The temperature strains in an arch structure are always to be considered carefully and their action must be provided for. While in stonework they may cause cracks or fissures, these follow

the lines of the mortar joints, and may perhaps distribute their effects on several of them, so as to do no serious damage. In concrete construction of the monolithic character, such as is usual for arches of smaller span, there are practically no joints, and cracks might be formed of irregular and uncertain location running out in thin edges which, with weather and frost action, might flake off, making unsightly or perhaps dangerous openings on the exterior and casting suspicion, at least, on the integrity of the interior. To provide for such a possibility in the construction of these

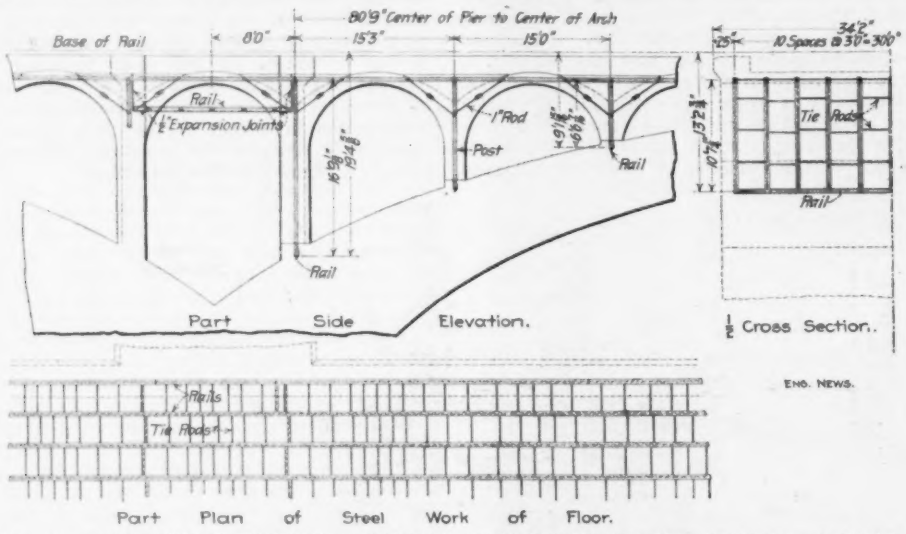


FIG. 4. PLAN AND ELEVATIONS, SHOWING REINFORCEMENT FOR SPANDREL ARCHES.

large arches, it was determined to build the main ribs as a series of voussoirs.

The piers and abutments were carried up above the springing line of the arches, and the voussoirs numbered. Nos. 1 and 1 were built as a portion of the pier or abutment. Then (the centering and molds being completed) the several voussoirs were built in the order in which they are numbered on the plan, voussoirs numbered 2 being located fourth in order from No. 1. Those numbered 3 being separated from those numbered 2 by three other sections; those numbered 4 being halfway between 1 and 2, and so on, as shown in Fig. 6. The succession was so arranged as to keep a symmetrical and as nearly as possible uniformly distributed load on the centering, and the number of voussoirs was planned so that at a certain stage of the work the arch would have alternate blocks completed, the keystone being one of the unfinished blocks.

The remaining blocks were then filled in, working toward the center, and the keystone being the last built. Each voussoir keys into the next by two projections on each side made by recesses built into the block first completed. This was done by securing planks of proper size on the face of the form dividing the mold into voussoirs. The blocks numbered 4 were held in place to prevent slipping until the blocks numbered 6 were built. No other blocks had any tendency toward movement. The face joints between voussoirs were made by securing strips of wood of triangular section to the molds, a sharp right-angled edge adjacent to a thin joint being avoided on account of the possibility of flaking or cracking off, should there be a very heavy strain on the extreme outer layer of concrete. The real joints are about 8 ft. apart, and intermediate false joints of similar style were made on the exterior faces of each voussoir. All the radial joints extend across the molding with which the top of the main arched rib is finished, and corresponding joints extend across the soffit from face to face of arch.

Additional provisions for expansion are made in the construction of the transverse spandrel arches, in the connections of the longitudinal rails by splices having some motion, but specially by the sliding arches and expansion joints at each pier and abutment.

Assuming that the piers and abutments are fixed, and that the arches must expand, a joint is made at each end of each arch, at its juncture with the adjacent immovable portion of the structure. There are six expansion joints (Fig. 7), essentially alike. They are intended to provide expansion or freedom of motion for that portion of the structure on which the moving load brings its immediate effect. It will be noted that the character of this portion of the structure differs materially from that of the lower portion of the bridge. In the latter every thing is massive and as nearly immovable as possible; in the former, by the combination of steel and concrete, an amount of elasticity is imparted to the structure which is designed to insure its durability under any and all conditions of exposure and service. The steel and concrete are proportioned to allow for the maximum range of expansion due to changes of temperature, also to absorb possible shocks due to derailments, or to the sudden setting of brakes on swiftly moving trains, and other similar conditions.

The section of concrete in the crown of the small transverse arches is purposely made a minimum, to provide for elastic expansion at this point. In addition to this, however, at each of the expansion joints referred to above, sufficient provision is made to cover any possible range of contraction and elongation due to changes in temperature for each arch. This expansion joint extends from the haunch of the main rib, to the level of the track, and is concealed and protected by the projection of the pier or abutment, where the spandrel arches recess into the same. There are two surfaces to this expansion joint, one surface being in a longitudinal plane with the extension of the face of the arch, and the other at right angles to it.

Fig. 5. View Showing Spandrel Arch Reinforcement.

The longitudinal surfaces both of pier and arch work are covered with heavy asbestos board, thus securing a sliding surface. The transverse joint is filled with several thicknesses of corrugated asbestos board, so that under pressure it could close, and when the pressure is relaxed could expand again, preventing entrance of foreign material into the joint. To assist in protecting this joint, the top is covered with a lead plate, which is folded into the joint, and filled with pitch or asphaltic cement, as shown in the plans.

These expansion joints are constructed one at each abutment, and two at each pier; the outermost spandrel arch at each end of each main arch can move slightly in a longitudinal direction at the piers or abutments. This movement is further provided for in the construction of the supporting blind arches built into each pier and abutment, which are fitted with sliding bearings, and with an expansion joint similarly packed with corrugated asbestos board, which joint extends across the whole width of the bridge floor, from coping to coping. This makes a section of floor approximating 15 ft. long, and the width of both trusses, which is separated by a longitudinal sliding joint, lined with asbestos board, between the fixed portions of pier and abutment copings, etc., and the central section of floor. The skeleton steel work in the floor is, however, made continuous across these sliding joints, and at the same time allowed to move longitudinally by incasing the transverse tie-rods with pipe which is buried in the concrete of the floor, thus permitting sufficient longitudinal movement for expansion or contraction. The skeleton iron work thus ties together the ends of piers, and holds them at the proper distance apart.

Having provided expansion joints not only on this bridge, but on several other structures, it may be interesting to state that careful observations up to date, indicate but very slight movement. On this bridge, brass expansion gages, each made in two parts, and graduated to read by means of verniers to thousandths of a foot, were set in the copings across each expansion joint, and careful readings of these gages were taken up to the completion of the work. The longest series covers four months from Jan. 20 to May 23, 1903, and was for the west side of the north span, this having been completed, and the coping laid first. The results of these readings show an extreme movement of 0.012-ft., of which 0.007-ft. was observed at the north end, and 0.005-ft. at the south end of span.

The maximum temperature of each day was noted, and the gage read simultaneously. It is evident that no accurate method of arriving at the average temperature of the whole mass of the structure could be devised, and that only general deductions might be made from such incomplete data. The record of gage observations does not indicate that concrete is extremely sensitive to heat and cold. While some irregularities of movement were noted, the gage readings showed contractions in cold and expansion in warm weather up to the limits as above stated of 0.012-ft. for 140-ft. span from temperature in January to that of May.

A series of accurate levels was taken on different parts of the bridge at different stages of construction, and covering a period of a little more than a month after the bridge was in full operation. The results show that practically no variations have taken place in the elevations of different portions of the structure. Repeated attempts have been made to measure a deflection of one of the arches under load, or to note a movement of expansion gage during the passage of heavy trains, but up to the present neither depression, expansion or contraction has been observable with the means at our disposal.

FALSEWORK, CENTERING AND METHODS OF CONSTRUCTION.

The concrete work was done under contract with Mr. G. H. Scribner, Jr., of Chicago, but this contract provided that the driving of additional piles for the foundations, the driving of all falsework piles for the support of centers, and the removal of the old bridge, should be done by the railway company's forces. Consequently, during a large portion of the work, two independent forces were engaged, one under the Superintendent of Bridges, and the other under the contractor. The former did everything to maintain traffic during progress of the work, considering the operation of its regular trains as of first importance, and sacrificing everything else thereto. For this reason the expense of the falsework was unusually heavy. Careful records kept during several months showed that for more than half

of the time the railway company's forces were laid out, and prevented from working by trains having the prior right of track.

The general plan of work was as follows: (1) To complete the driving of piles for additional foundations for the south pier, the north pier, the south abutment and then the north abutment. (2) To drive falsework piles for the north span, then for the south span and then the center span. (3) To raise the original iron bridge 5 ft. on account of change in grade. This was done by means of jacks and cribbing timbers on the piers. The raising of the old spans to this elevation permitted the construction of the main rib of each arch underneath the old bridge. The contractor for concrete work was to furnish and erect all centering and molds required, also to construct the timber and sheeting of all foundation cofferdams. The plans for centering were prepared by the railway company, and are shown in detail in Figs. 8 and 9.

In building the north and south arches, this plan was followed exactly, but for the center arch, to provide for the possibility of high water which might bring down large amounts of drift, five 60-ft. deck plate girder spans resting on pile piers were used over the deepest portion of the stream, the tops of the girders finishing essentially at the same elevation as the pile caps in the other arches, and the same centering was used, supporting it upon the plate girder spans referred to. A platform was constructed large enough to lay out in full one-half the arch, and the centering was laid out and framed by patterns made on this platform.

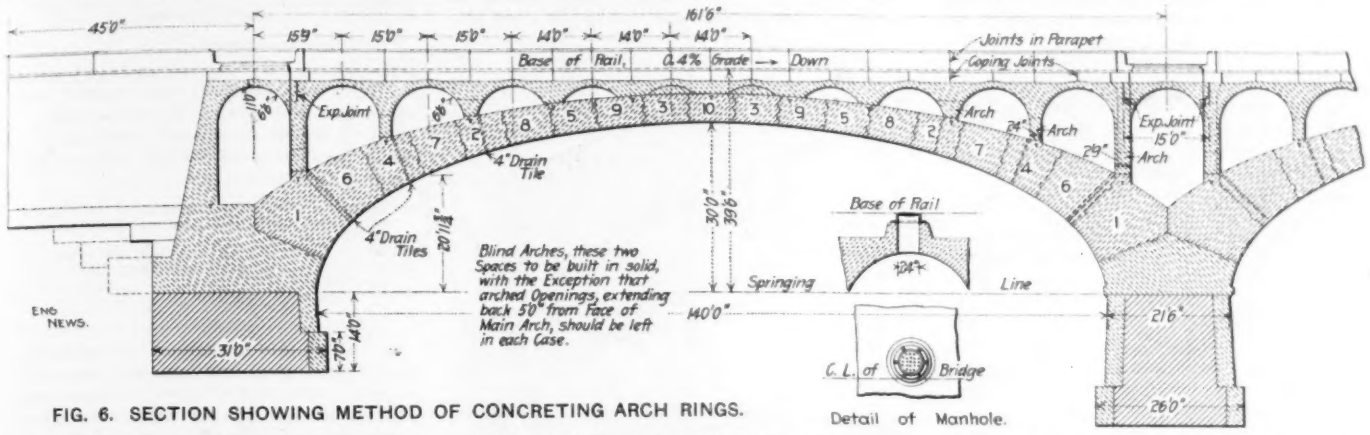


FIG. 6. SECTION SHOWING METHOD OF CONCRETING ARCH RINGS.

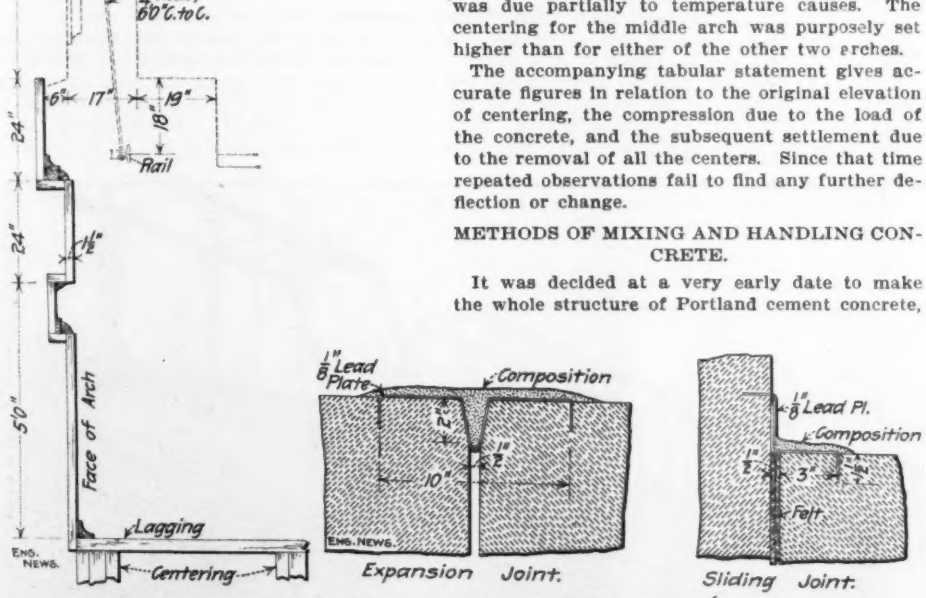


FIG. 7. DETAILS OF EXPANSION JOINTS.

Enough centering was made to erect two complete arches, and about half the centers were left in place in the north and south arches, while the remainder were being used for the construction of the center arch. Three full sets of arch sheeting were required. As shown by the plans, folding wedges were provided, by means of which the centers could be struck after the concrete was completed. Each completed main arch rib was estimated to bring a load of nearly 3,000 tons on the centering, or approximately 18 to 20 tons per pile in the falsework.

The centering of the north and south arches was set up a little over 1 in. above the calculated elevation for finished concrete work to allow for compression and settlement of falsework. This was found to be insufficient in case of the north arch, although it is probable that the difference observed later in the final elevation of north arch was due partially to temperature causes. The centering for the middle arch was purposely set higher than for either of the other two arches.

The accompanying tabular statement gives accurate figures in relation to the original elevation of centering, the compression due to the load of the concrete, and the subsequent settlement due to the removal of all the centers. Since that time repeated observations fail to find any further deflection or change.

METHODS OF MIXING AND HANDLING CONCRETE.

It was decided at a very early date to make the whole structure of Portland cement concrete, not using any of the so-called natural hydraulic cements. The contract for the work being closed late in the summer, it would be impossible to get any large portion of the work completed before cold weather, and by using Portland cement concrete it might be possible to work continuously during the coldest weather.

The massive character of the work, the large amount of concrete required for each foundation and the nature of this foundation work, also the fact that the remainder of the work would divide naturally into large monolithic masses, were conditions calling for the use of the best available cement. It was not a case where economy could be practiced by using a cheaper natural hydraulic cement. Special care was taken, however, to modify the mixtures in order to keep the expense as low as possible by varying the proportion of the cement, sand and crushed stone in accordance with the character of service to which the structure might be subjected.

Referring to the plans, it will be noted what proportions were used for different parts of the foundations, of the piers, of the main arches, of the spandrel arches, copings, etc. The general principle was that of making the center of each large mass of a cheaper or less expensive grade of concrete, reserving the richer mixtures for use where strains might alternate from compression to tension. Steel rails are embedded in some of the foundations in order to tie together and give additional tensile bond where the old foundations had been in bad condition.

The "Owl" brand of Portland cement, manufactured by the German-American Portland Cement Works, of La Salle, Ill., was used solely in the construction of this bridge. The following table is a summary of one of the series of tests made by the railway company's inspector. The uniformity of the results was due not only to careful manipulation but to unusual evenness of the cement. The set of tests quoted was not made especially for this record, but is one of several complete sets of lists copied from our record book.

Tests of "Owl" Portland Cement, made at Laboratory of Illinois Central R. R.

Fineness: No. 50 sieve, 100%; No. 100 sieve, 98%. Proportions: Cement, 23⅓%, standard sand, 21%; water, 16⅔% for neat, 9⅝% for 1 to 1, 8⁷/₁₁% for 1 to 2, and 8½% for 1 to 3. Time of setting: 6 hrs. for neat, 4 hrs. for 1 to 1, 3 hrs. for 1 to 2, and for 1 to 3.

	Tensile strength, lbs. per sq. in.			
	Neat.	1 to 1.	1 to 2.	1 to 3.
1 day	154.1	149.1	70.1	30.2
7 days	611.9	536.1	348.3	209.8
28 "	791.3	707.6	457.1	304.6
3 months	909.4	737.0	635.9	345.5
6 "	937.8	834.7	666.1	387.9
12 "	1,031.2	949.7	688.5	417.5

Deflection of Arches of Big Muddy Bridge.

Elevation taken on under side of keys of main arches.

Conditions of work when elevations were taken.	North arch		Center arch		South arch	
	West side.	East side.	West side.	East side.	West side.	East side.
Centers ready for concrete	385.06	385.06	385.95	385.93	386.32	386.33
Concrete finished in main arches	384.92	384.92	385.77	385.70	386.21	386.22
Compression, etc., of centers	0.14	0.14	0.18	0.14	0.11	0.11
Alternate centers removed	384.92	384.92	385.77	385.70	386.12	386.15
Further yield of centers	0.00	0.00	0.00	0.00	0.09	0.07
Rest of centers removed	384.69	384.70	385.70	385.71	386.03	386.06
Deflection of arches*	0.23	0.22	0.07	0.08	0.09	0.09
Total change from original height of centers.†	0.37	0.36	0.25	0.23	0.29	0.27
Figured elevations on plan	384.95	384.95	385.60	385.60	386.24	386.24
Allowed for compression, shrinkage, etc.‡.....	0 11	0.11	0.35	0.33	0.08	0.09
Final elevations as compared with original plan	—0.26	—0.25	+0.10	+0.10	—0.21	—0.18

*The greater apparent deflection of the north arch includes all changes due to the variation of 50° to 70° in temperature, from July 25, 1902, to Jan. 19, 1903.

†The centers were loosened on dates as follows: North arch, Aug. 8-15, 1902, wedges were loosened; when removed Jan. 19, 1903, there was no further deflection of the arch; rib of arch was finished July 25, 1902. South arch was finished Oct. 5, 1902, wedges were out Jan. 15, 1903. Center arch, finished Dec. 5, 1902, wedges were out Jan. 15, 1903. Note.—The bridge is on a grade of 0.004%, rising to the south.

‡The centers for the middle arch were set high purposely; 1st. To make this arch higher at the crown than its position on the grade; and, 2d. To allow for a slight movement of the north pier, which had apparently yielded under the thrust of the north arch, which was finished first, centers being loosened before any centering was in place in the center arch, or anything in place to assist this pier in resisting the thrust.

The standing rule as to testing cements was that cement should not be used from any given car until said car had been sampled, tested and accepted. To save time it was frequently arranged that certain large bins at the cement works should be reserved, from which cars should be shipped to the work; the cement in these bins being sampled, tested and accepted. It was occasionally found that 24-hour tests showed too

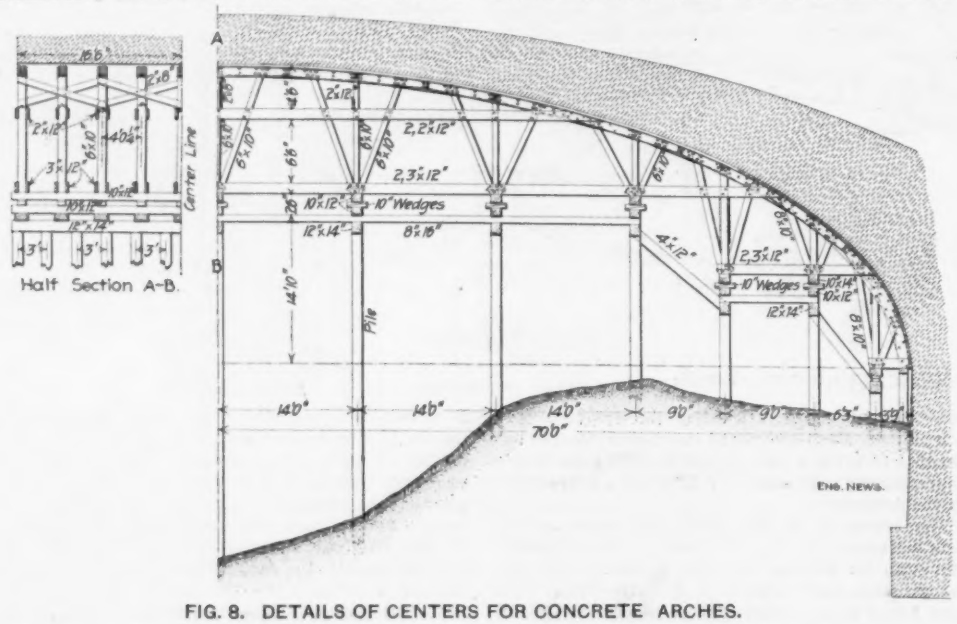


FIG. 8. DETAILS OF CENTERS FOR CONCRETE ARCHES.

little tensile strength, but this was usually corrected by the 3-day and 7-day tests, and out of 15,000 barrels of cement used none was rejected.

Nearly all the concrete used in the structure was mixed by machines. These were set up, taken down, moved and set up again as occasion demanded, generally with the idea of delivering the mixed concrete as near as possible to the point where it should go into the work.

For the foundation work, crushed stone and sand were generally delivered through the floor of the old bridge, to convenient points near each pier or abutment, where the concrete was mixed either by hand or by machinery, the mixed concrete being delivered by wheelbarrows from runplanks, and shoveled and tamped into place. Derricks were used,, handling tubs of about 1 cu. yd. capacity for delivering concrete from the mixers for the voussoirs of main arches. Sometimes these buckets were dumped, the mixed concrete reloaded into barrows and thus delivered where wanted. One-half of each main arch had to be constructed underneath the old truss bridge, this work being in progress while the regular traffic of the road was being carried on the structure. All this necessitated extra handling of concrete, placing and ramming it in confined positions not easy of access.

The concrete work on the west half of the spandrel arches was also handled by derricks, but on the east half (the old spans having then been removed, and a temporary track pushed out over the skeleton steel work), a concrete mixer was mounted on flat cars. The cars of material were delivered next to the car carrying the mixer, and elevated wheelbarrow runs were used for delivering the materials to the mixer.

In general a fairly wet concrete was used for all this construction. But very little ramming was necessary to bring it to the quaking condition. All exposed work is faced with mortar, which was intended to be 1½ ins. thick. This was generally made of too fluid a nature to give the desired thickness, but the use of even a small amount of facing mortar has probably secured a finer finish than could have been made by spading or similar methods.

All exposed horizontal surfaces such as copings received a hard trowel finish in the usual style of sidewalk work. No attempt was made to remove or cover over the usual marks made by the planks of the forms, and only a very small amount of pointing was done on the exterior surfaces. In general the use of facing mortar made pointing unnecessary.

COST.

It has been stated already that the cost of the work done directly by the railway company was excessive because of making the movement of trains the matter of primary consideration. It is also the fact that in many ways the work performed by the company's men was made more expensive on account of changes in the order or in the character of the work, made to facilitate the portions to be done by the contractor. In this way expense was sacrificed to save time.

It is to be regretted that no accurate account was kept of the cost of the several parts of the work done by the railway company's men. They were, however, frequently called away to attend to emergency cases on our own or adjacent lines, and the keeping of an accurate distribution of the cost of labor on different portions of the work was lost sight of. The work performed by them included the following: Furnishing and driving piles required for foundations and falseworks; framing and capping falsework piles and placing the heavy timbers beneath the centers; raising the old bridge 5 ft. on blocking, and finally taking it down; removing the falsework and clearing up about the structure, and the laying of temporary track on the west half of the bridge. Besides this, the removal of a portion of the south abutment, and the gathering up and removal of scattered bridge stone around the piers and abutments, handling the drift at high water, and similar work, were additional causes of expense.

Cost of above work by railway company.....	$54,402
Amount paid to the contractor	72,128
Skeleton steel work (excluding erection).....	4,451
Crushed stone	7,500
Engineering and sundry expenses	6,700
Total cost, approximately	$125,000

The above may be compared with the cost of enlarging or rebuilding the original piers and abutments and placing thereon a superstructure of three double-track steel spans built to carry the present standard loading, which may be estimated as follows:

Masonry, covering reconstruction of both abutments and lengthening piers, say 2,500 cu. yds. at $12 a yd	$30,000
Falsework, removing old, erecting new spans....	10,000
Shop work, 3 double-track spans each 158 ft long, 2,150,000 lbs., at 3.58 cts.....	75,180
Erection, painting and floor	10,500
Total $125,680	

In making this comparison, the masonry for the double-track steel structure has been assumed as though constructed of stone corresponding to the existing work in the old bridge. An estimate for using concrete instead of stone masonry would probably show a less total cost, but would make a much less satisfactory structure. Both of the old abutments were in such bad condition that it is probable they would have needed entire reconstruction for a double-track steel bridge.

While it is impossible to give the detailed cost of the several parts of the work, it may be interesting to note the following: There were upwards of 5,000 cu. yds. of excavation for foundations, costing an average of 76 cts. per yd.; there were over 12,000 yds. of concrete of various grades costing from about $4 to $10 per yd., and averaging $5.43 per yd.; there were a little over 400,000 ft. B. M. of timber used in cofferdams and centering, forms, etc., and about 300,000 lbs. of steel in the skeleton construction, supporting the floor of the bridge; this steel, complete in place, cost 1.81 cts. per lb. Of this amount, the steel rail itself is estimated at 1.2 cts. per lb., showing the cost of handling, punching, fitting, and erecting to be 0.61-ct. per lb.

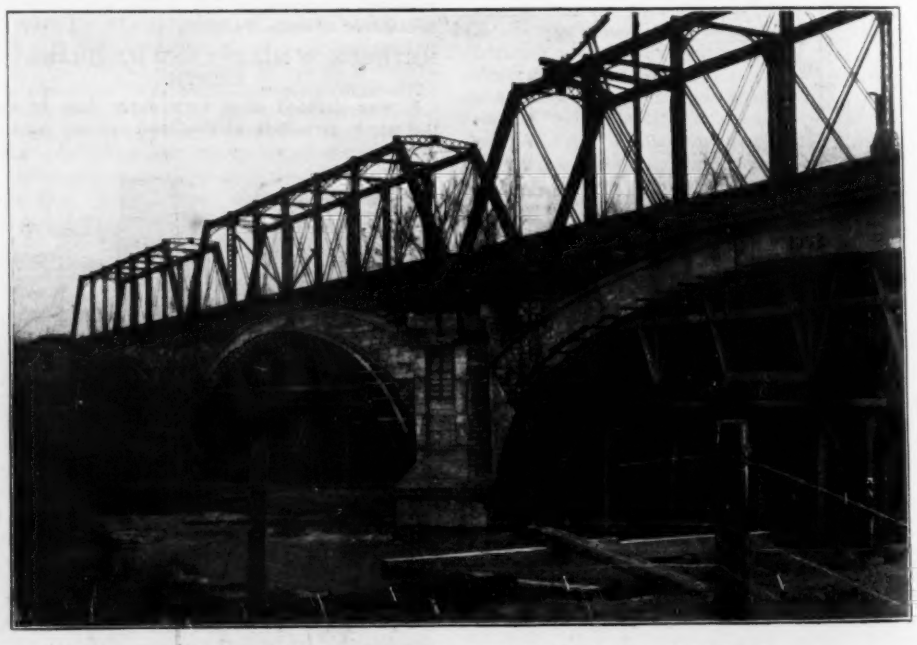


FIG. 9. GENERAL VIEW OF ARCH CENTERS IN PLACE.

ENGINEERS AND CONTRACTORS.

The contractor for foundations and concrete work, not including furnishing and driving piles, was Mr. G. H. Scribner, Jr., Chicago, Ill. The pile-driving, maintenance of track, raising and removing old spans was all done by the Bridge Department of the Illinois Central R. R., under Mr. O. J. Travis, Superintendent of Bridges. The engineering work in the field was under the charge of Mr. H. H. Hadsall, Chief Masonry Inspector,

FIG. 1. SIDE VIEW OF GOOD TYPE OF CALIFORNIA WELL RIG.

(Derrick with mast 40 ft. high, with 10-in. sand pump and jars just pulled out of 12-in. well. Twenty-five horse-power boiler mounted on separate trucks.)

and Mr. L. W. Hancock, Resident Inspector, with two or three assistants. The office engineering work was under the charge of Mr. R. E. Gaut, Chief Draftsman, Bridge and Building Department; Mr. C. C. Chandler, in charge of the structural steel work, and Mr. J. B. Gaut, Shop Inspector. The writer, as Engineer of Bridges and Buildings of the Illinois Central R. R., had general charge of the entire work, and is indebted to all of the above-named for the successful completion of the work.

THE CALIFORNIA OR "STOVE-PIPE" METHOD OF WELL CONSTRUCTION FOR WATER SUPPLY.

By Charles S. Slichter.*

The peculiar conditions of water supply existing in Southern California have led to the development of a special type of well, which the writer believes to be admirably adapted to conditions found in many places in the eastern part of the United States. It is the object of this article to call the attention of those interested in recovering ground water in large quantities to the many points of excellence of the California type of well and method of well construction.

The valleys in Southern California are filled with deposits of mountain debris, gravels, sands, boulders, clays, etc., to a depth of several hundred feet, into which a considerable part of the run-off of the mountains sinks. The development of irrigation upon these valleys soon became so extensive that it was necessary to supplement more and more the perennial flow of the canon streams by ground water drawn from wells in the gravels. This necessity was greatly accentuated by a series of dry years, so that ground waters became a most valuable source of auxiliary supply for irrigation in the important citrus areas in Southern California. The type of well that came to the front and developed under these circumstances is locally known as the "stove-pipe" well. It seems to suit admirably the conditions prevailing in Southern California. In developing water supply for irrigation the item of cost is, of course, much more strongly emphasized than in developments for municipal supply. The drillers of wells in California were not only confronted with a material which is almost everywhere full of boulders and like kinds of mountain debris, but also by a high cost of labor and of well casings. It was, undoubtedly, these difficulties that led to the very general adoption in California of the stove-pipe well.

The wells are put down in the gravel and boulder mountain outwash, or other unconsolidated material, to any of the depths common in other localities. One string of casing, in a favorable location, has been put down over 1,300 ft. The usual sizes of casings are 8, 10, 12, 14 ins., and even larger. A common size is 12 ins. The well casing consists of: first, a riveted sheet steel "starter," from 15 ft. to 25 ft. long, made of two or three thicknesses of Number 10 sheet steel, with a forged steel shoe at the lower end. In Fig. 3, a starter is shown resting on two short lengths of casing. This starter is 21 ft. long, made of double No. 10 sheet steel, with a ¾ × 8 × 12-in. steel shoe riveted on the bottom. In ground where large boulders are encountered, these starters are made heavier, the shoe 1 in. thick and 12 ins. deep, and three-ply instead of two-ply No. 10 sheet steel body.

Above the starter, the rest of the well casing consists of two thicknesses of No. 12 sheet steel made into riveted lengths, each 2 ft. long. One set of sections is made just enough smaller than the other so that one length will telescope snugly inside of the other. Each outside section overlaps the inside section 1 foot, so that a smooth surface results both outside and inside of the well when the casing is in place; and so that the break in the joints is always opposite the middle of a 2-ft. length. It is these short, overlapping sections which are popularly known as "stove-piping." A pile of this casing ready for use is shown in Fig. 1 and several lengths are shown on a larger scale in Fig. 3.

The casing is sunk by large steam machinery of the usual oil well type, but with certain very important modifications. In ordinary material the "sand-pump" or "sand bucket" is relied upon to loosen and remove the material from the inside of the casing. The casing itself is forced down, length by length, by two hydraulic jacks, buried in the ground and anchored to two timbers 14 × 14 to 16 ins., which are planked over and buried in 9 or 10 ft. of soil. These jacks press upon the upper sections of the stove-piping by means of a suitable head. In Fig. 3, the clevises of the pistons of the hydraulic jacks can be seen hooked over the ears of the well cap. The jacks, shown in the cuts, have 8-in. piston and 4½-ft. stroke and a combined pull of about 120 tons. The driller who stands at the front of the rig has complete control of the engine, hydraulic pump and valves by which pistons are moved up or down and also of the lever which controls two

FIG. 2. REAR VIEW OF WELL RIG, SHOWING ENGINE AND HYDRAULIC PUMP.

(When ready to move, mast swings backward on hinges with top resting on stand shown at rear end of rig. Jackscrews are placed under sills and the whole machine is raised sufficiently to allow wheels to be placed on axles bolted firmly to sills. One of the axles is shown at rear of rig covered with sack.)

*Consulting Engineer, U. S. Geological Survey, Madison, Wis.

clutches which cause tools to work upon and down or to be hoisted.

The sand pumps used are unusually large and heavy. For 12-in. work they will vary in length from 12 to 16 ft., 10⅝ ins. in diameter, and will weigh, with lower half of jars, from 1,100 to 1,400 lbs.

After the well has been forced to the required depth, a cutting knife is lowered into the well and vertical slits are cut in the casing where desired. A record of material encountered in digging the well is kept and the perforations are made opposite such water-bearing strata as may be most advantageously drawn upon. A well 500 ft. deep may possess 400 ft. or more of screen if circumstances justify it.

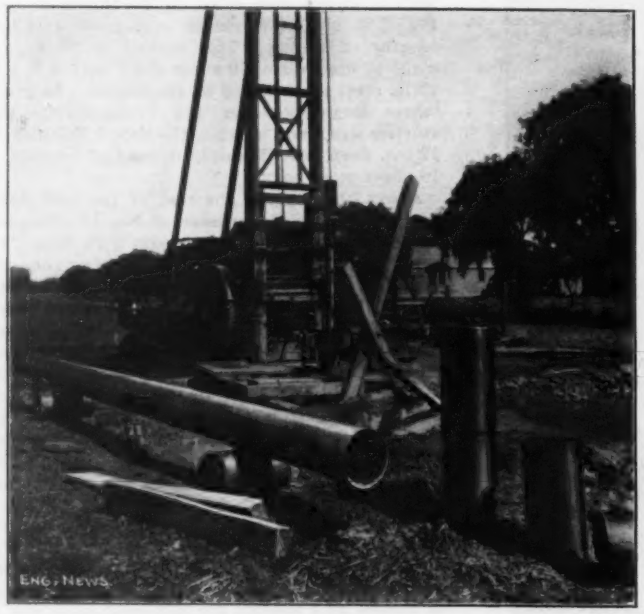


Fig. 3. View Showing a 12-In. Stove Pipe Starter.

(This starter is 21 ft. long, made of double, No. 10 sheet steel, with ¾ × 8 × 12-in. steel shoe riveted on end. A few lengths of stove pipe casing appear at the right of the cut. Immediately in front of the rig can be seen the clevises of pistons of hydraulic jacks hooked over ears of well cap.)

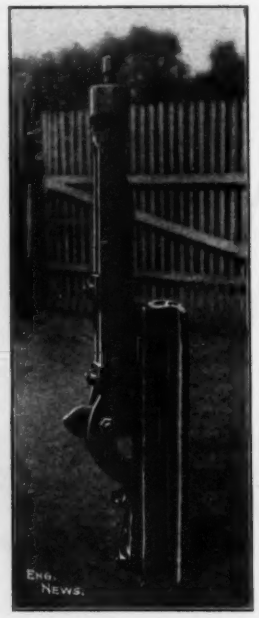


Fig. 4. View of Perforator for Slitting Stove Pipe Casing After Well is Completed, Shown in Cutting Position with Knife Extended.

(The perforator is handled with 3-in. standard pipe with ¾-in. standard pipe inside. In going down or coming out of well the weight of ¾-in. line holds point of knife up. When ready to stick, the ¾-in. line is raised, at same time raising on 3-in. line with hydraulic jacks.)

Fig. 4 shows the perforator for slitting stove-pipe casing. It is handled with 3-in. standard pipe with ¾-in. standard pipe on the inside. The perforator is shown in cutting position with knife extended. In going down or in coming out of the well the weight of ¾-in. line holds point of knife up. When ready to "stick" the ¾-in. line is raised. By raising slowly on 3-in. line with hydraulic jacks, cuts are made from ⅜-in. to ¾-in. wide and from 6 ins. to 12 ins. long, according to the material at that particular depth.

Fig. 5 shows another type of perforating knife. The revolving cutter punches five holes at each revolution of the wheel. This style of perforator is called a "rolling knife." By means of this tool 60 ft. of perforations were cut in the well shown in Fig. 7.

A great many different kinds of perforators are in use in California; in fact, the perforator is a favorite hobby of local inventors. The different patterns in use seem to work well. Those shown in the photographs are very good.

The advantages of this method of well construction are quite obvious. For wells in unconsolidated material, the California type is undoubtedly the best yet devised. I believe that wells of this type would be highly successful in the unconsolidated coastal deposits on Long Island, New Jersey, and similar localities. The absence of boulders and very coarse gravels in those deposits might possibly make it more advantageous to use the hydraulic jet instead of the ponderous sand bucket in soft material, but this is the only modification that Eastern conditions seem to suggest.

Among the special advantages in the stove-pipe construction we may enumerate the following:

(1) The absence of screw joints liable to break and give out.

(2) The flush outer surface of the casing without couplings to catch on boulders, or hang in clay.

(3) The elastic character of the casing, permitting it to adjust itself in direction and otherwise to dangerous stresses, to obstacles, etc.

(4) The absence of screen or perforation in any part of the casing when first put down, permitting the easy use of sand pump and the penetration of sand, etc., without loss of well.

(5) The cheapness of large size casings, because made of riveted sheet steel.

(6) The advantage of short sections, permitting use of hydraulic jacks in forcing casing through the ground.

(7) The ability to perforate the casing at any level at pleasure is a decided advantage over other construction. Deep wells with much screen may thus be heavily drawn upon with little loss of suction head.

(8) The character of the perforations made by the cutting knife are the best possible for the delivery of water and avoidance of clogging. The large side of the perforation is inward, so casing is not likely to clog with silt and debris.

(9) The large size of casing permitted in this system permits a well to be put down in boulder wash where a common well could not possibly be driven.

(10) The uniform pressure exerted by the hydraulic jacks is a great advantage in safety and in convenience and speed over any system relying upon, driving the casing by a weight or ram.

(11) The cost of construction is kept a minimum by the limited amount of labor required to man the rig as well as by the good rate of progress possible in what would be considered in many places impossible material to drive in, and by the cheap form of casing

An idea of the cost of construction of these wells can best be given by quoting actual prices on some recent construction in California. According to contracts recently let near Los Angeles, the cost of 12-in. wells was: 50 cts. per foot for the first 100 ft., and 25 cts. additional per foot for each succeeding 50 ft., casing to be furnished free. This makes the cost of a 500-ft. well $700 in addition to casing. The usual type of 12-gage, double, stove-pipe casing is about $1.05 a foot, with $40 for 12-ft. starter with ¾ × 8-in. steel ring. A good driller is paid $5 per day, helpers $2.50 per day. The cost of drilling will run higher than that given above in localities where large and numerous boulders are encountered.

The photographs, Figs 1, 2 and 3, are of a new rig of a very excellent type owned by Mr. H. W. Riggle, 922 West Jefferson St., Los Angeles. The drillers build their own rigs according to their own ideas, so that no two rigs are exactly alike—that is, the drillers pick out the castings and working parts and mount them according to ideas that experience has taught them are the best for the wash formations in which they must work.

In Fig. 7 is shown another excellent California rig owned by Mr. J. B. Proctor, of Compton, Los Angeles Co. As it appears in the cut it has just finished a 12-in. well which flows 404 miners inches, or about 5,250,000 gallons, per 24 hours. This well is 848 ft. deep and has about 60 ft. of perforations. Mr. Proctor used four hydraulic jacks in sinking this well, developing a pressure of from 160 to 200 tons.

It is not very profitable to name individual wells of this type and give yield of same, since conditions vary so much from place to place. From the method of construction it must be evident that this type of well is designed to give the very maximum yield, as every water-bearing stratum can be drawn upon. The yield from a number of wells in California of average depth of about 250 ft., pumped by centrifugal pumps, varied from about 25 to 150 miners' inches, or from 300,000 to 2,000,000 gallons a day. These are actual measured yields of water used for irrigation.

Among the very best flowing wells in Southern California are those near Long Beach. The Boughton well, the Bixby wells and the wells of the Sea Side Water Co. are 12-in. wells varying from 500 to 700 ft. in depth and flowing about 250 miners inches each, or over 3,000,000 gallons per 24 hours. The well shown in Fig. 7 is located about four miles northeast of Long Beach and its flow is the greatest I have seen reported.

Among the records of depth are those of 1,360 ft. for 10-in. well and of 915 ft. for 12-in. well.

Fig. 5. View of Roller Type Perforator.

Mr. Proctor is now working on a 14-in. well which has already reached a depth of 704 ft.

THE USE OF DISTANT SIGNALS AT INTERLOCKING PLANTS.*

This committee, consisting of Messrs. Shaver (chairman), Elliott, Pflasterer, Peabody, and Goodman, held a meeting at Chicago, April 7. In the opinion of the committee a distant signal at an interlocking plant cannot be worked automatically with success, and therefore they consider the question before them to be, "When, under

*Report of a Committee at the Annual Meeting of the Railway Signal Club.

Results of Circular Inquiries Respecting Practice in Operation of Distant Signals.

Railroad	Answers to Questions. 1	2	3	4 A	4 B Max. ft.	4 B Min. ft.	4 C Max. ft.	4 C Min. ft.
[illegible] & Albany	Yes	Yes	No	None	...	...	...	...
[illegible]an Pacific	Yes	Yes[1]	No[2]	None	...	...	...	...
[illegible] & Alton	Yes	Yes	No[2]	None	...	...	...	...
[illegible] & E. I.	Yes[1]	Yes[1]	No	None	...	...	...	...
[illegible]. B. & Q.	Yes[1]	Yes[1]	No[2]	None	...	...	...	...
[illegible]land, C., C. & St. L.	Yes[1]	Yes[1]	No[2]	None	...	...	...	...
[illegible]go, G. W.	No	No	...	2	...	...	...	...
[illegible]go, H. & D.	Yes[1]	Yes[1]	No	None	...	...	...	...
[illegible]go, M. & N. W.	Yes	Yes	Yes[1]	8	1500	1200	2000	1700
[illegible]go, St. P., M. & O.	Yes	Yes	Yes	3	2000	1200	3400	1500
[illegible]go, Mil. & St. P.	Yes	Yes	Yes	9	3200	600	4100	900
[illegible]ouri, K. & T.	Yes	Yes	Yes	2	2775	2500	3575	2200
[illegible]ouri, N. O. & T. P.	Yes	Yes	Yes	45	2000	...	2250	...
[illegible]ouri, L. & W.	Yes[1]	Yes[1]	Yes	4	4850	1500	5000	2600
[illegible] Central	Yes	Yes	No[2]	None	...	...	...	...
[illegible] Shore & M. R.	Yes	Yes	Yes	4	2581	1450	3084	2484
[illegible] Valley	Yes	Yes	Yes	Not given	2500	2000	...	...
[illegible]land	Yes	Yes	...	18	3200	1700	2700	1500
[illegible]go Central	Yes	Yes[1]	Yes[1]	20	3000	2500	3500	3000
[illegible] York, N. H. & H.	Yes	...	No	None	...	...	...	...
[illegible] York Cent. & H. R.	Yes	Yes	Yes[1]	...	...	...	...	...
[illegible]rn Pacific	...	...	No	None	...	...	...	...
[illegible]sylvania[3]	Yes	Yes	Yes[1]	8	2500	...	3500	...
[illegible]sylvania Lines West	Yes	Yes	Yes	25	6000	1500	6000	1500
[illegible]re & L. E.	Yes	Yes	Yes	6	1600	1600	3000	2000
[illegible]ro Pacific	Yes[1]	Yes[1]	No[2]	None	...	...	...	...
[illegible]n Pacific	Yes	Yes	No[2]	None	...	...	...	...
[illegible]ash Line	Yes	Yes	No[2]	None	...	...	...	...

[1] In some cases. [2] Some installations planned or contracted for are to have power operated distant signals. [3] Approximate distances. [4] Average distances.

various conditions, should distant signals at interlocking plants be mechanically operated, and when power operated?" A list of questions was sent out, and replies were received from 28 railroads. These questions were as below, and the replies are shown in the table above.

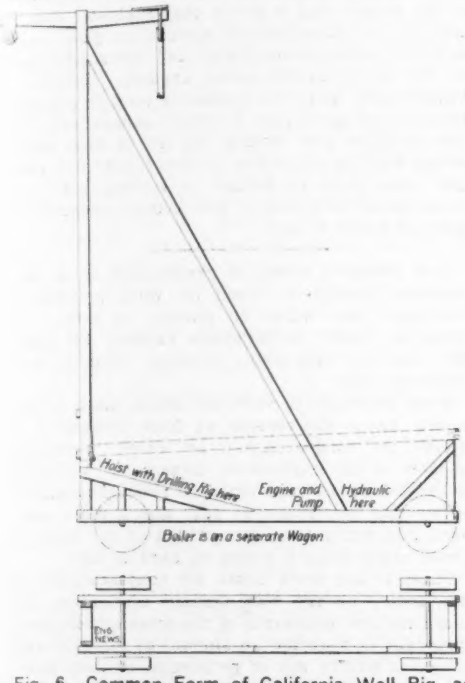


Fig. 6. Common Form of California Well Rig, as Designed by Fulton Engine Works, Los Angeles, Cal.

1. Do you approve of power-operated Distant Signals?
2. Do you propose using power-operated Distant Signals in future installations?
3. Have recent new installations of interlocking plants on your road been provided with power-operated Distant Signals?
4. (a) How many power-operated Distant Signals have you in use? (b) If any, at what distance are they located from Home Signal? (c) From tower?

Twenty-five roads look with favor on the power-operated distant signal. Only one road has had an unfavorable experience and prefers mechanical signals. Seventeen roads expect to use power-operated distant signals; one road declares the usefulness of the distant signal to be still an open question, and one, on which there is a mechanical distant signal 3,840 ft. from the tower, seems to regard mechanical signals as satisfactory; the question of power-operated distant signals has not been considered.

The committee, instead of going into a discussion of points which have already been considered by the club, refers to the paper by Mr. Elliott, and the letter by Mr. E. M. Herr, which were read at the last annual meeting, and are printed in the "Proceedings," pages 76 and 85. (Eng. News, Nov. 13, 1902, p. 410.) In conclusion, the report says:

A distant signal is a fixed signal used in connection with a home signal to regulate the approach thereto, and should be so placed that the fastest train to be controlled can be stopped easily, under the worst conditions, between it and the home signal. To fulfil these conditions safely and successfully where high speed is maintained, the power-operated distant signal must be used. Where trains run at low speeds, the distant signal may be placed nearer to the home signal, but even in such cases this committee does not recommend that the signals be mechanically operated unless it be by a pipe line properly installed.

COVERING AN OLD RESERVOIR AT BROOKLINE, MASS.

By F. I. Winslow.*

Although in the heart of the Metropolitan Water District the town of Brookline has clung to her own water system, the supply for which is obtained from driven wells adjacent to the Charles River in West Roxbury, within the limits of the city of Boston. There has been for years on Fisher Hill, Brookline, an open reservoir, but this was abandoned ten years ago, on account of trouble caused by vegetable growths, and could not be used again until it had been covered. In the meantime a small covered reservoir (see Eng. News, Dec. 21, 1893) had been built and placed in service. Early last spring an appropriation was obtained and on March 20 the work of covering the old reservoir was begun.

The old reservoir was somewhat irregular in plan. Two of its sides were nearly parallel, but slightly curved; one end was gently curved, and the other was at an angle of about 45° with the two sides. These outlines were brought to a rectangular form by straightening the sides and one end and by enlarging the reservoir at the other end. The reservoir was in excavation and embankment. Its bottom was covered with 12 ins. of puddle and 4 ins. of gravel concrete, and the inner slopes with 18 ins. of puddle, 12 ins. of broken stone and 15 ins. of hand-laid stone. As originally built, the reservoir was about 160 ft. wide and had an average length of about 360 ft. The side slopes were 1½ to 1 and the water depth was 20 ft. The storage capacity was about 6,000,000 gallons. The net capacity of the enlarged and covered reservoir will be about 7,000,000 gallons.

For covering, the groined arch, formed to a true ellipse, with a chord of 12 ft. and a rise of 2½ ft., was adopted. This has been made of concrete, 6 ins. thick at the crown, supported by 20-in. piers, 13.67 ft. c. to c., carrying 30 tons each. About half of this load is due to the concrete roof. The piers are of second quality brick, but rest on concrete footing extending above and below the bottom of the reservoir, as shown in the accompanying section. The section also shows the design adopted for supporting the groined arches over the side slopes, where the piers rest on three low walls, between which the sloping embankment was removed to form a level stretch of floor. An exception to this method is found at the south end of the reservoir, where for a part of the distance a wall 9 ft. thick at the base and 25 ft. high was built from the stone taken from the old slopes. The piers and walls rest on hardpan.

*Assistant Engineer, City Engineer's Office, Boston, Mass.

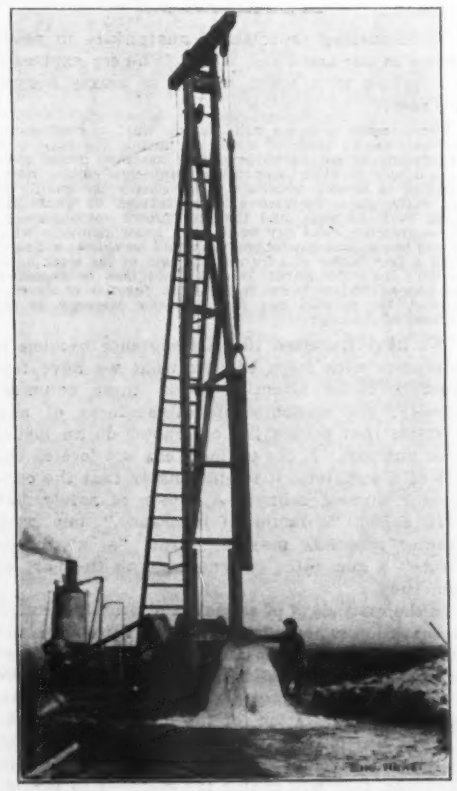
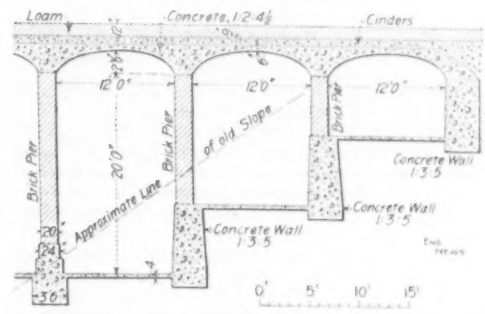


Fig. 7. California Well Rig After Completing 12-in. Well Flowing 404 Miner's Inches, or 5,250,000 Gallons per 24 Hours.

As shown on the section, the new concrete floor beneath the old slopes is 4 ins. thick. The same thickness of concrete was added to the old 4-in. concrete floor on the bottom of the reservoir.

Atlas, Dragon and Vulcanite cements were used, and the concrete was mixed in the proportion of 1:2:4½ for the roof and 1:3:5 for walls. The stone was specified to pass through a 3-in. mesh.

The concrete for the walls and the bases of the piers was mixed very wet and not rammed, while that for the roof was mixed drier and thoroughly rammed. The summits of the arches were waterproofed and drains were laid from between the manholes to the sides of the reservoir. All the water surfaces are now being plastered, and it is expected that the water will be let into the reservoir by the end of the present season.



Section Through Groined Arch Concrete Roof of Brookline Reservoir, Over the Embankment.

The method of centering was by the use of a 6 × 6-in. center post where the apices of the four triangular forms for the roof were placed. The base of the forms rested on similar posts at the corners, and the rectangular forms between the groined arches were firmly bolted together with the object of preventing any possible movement of the groined arch forms while concrete was being rammed above them.

A cableway, stretching entirely across the work lengthwise, was employed to convey all material. A Smith mechanical concrete-mixer was placed at the engine end of the cableways.

Fifteen manholes were placed on the center line of the reservoir, an extra large one being placed in the southeast corner for use in case of repairs.

A weir overflow, 6 ft. wide, was provided, discharging into a 24-in. pipe. A 24-in. pipe serves as inlet and outlet. A second inlet, 16 ins. in diameter, is also provided, and a 10-in. drain serves to empty the reservoir when required.

The entire work was designed and superintended by Mr. Fayette F. Forbes, C. E., Superintendent of the Brookline Water-Works, and Past President of the New England Water-Works Association. No contract for labor was let. The force hired by Mr. Forbes worked 52 hours per week. The cost of this work will be about $80,000.*

(*In Engineering News of Sept. 17, 1903, on p. 238, will be found a table compiled by Mr. Leonard Metcalf, Assoc. M. Am. Soc. C. E., giving particulars relating to twelve other reservoirs or filters having groined arch coverings of concrete, and built in the United States from 1895 to 1903, inclusive. The pier load on these twelve reservoirs ranged from 8.9 to 17.7 tons per sq. ft., based on the total load above the springing line of the arches and the minimum section of the pier. On the same page of Engineering News there is a brief description of a roof covering (also included in the table) built this year at Natick, Mass., with Mr. Frank L. Fuller, M. Am. Soc. C. E., of Boston, as engineer. This roof was also built over an old reservoir; the roof over the old slopes abutted directly on the granite paving of the upper part of the slope, while below one row of piers was built into the embankment. —Ed.)

SLAG SAND is being produced at the Soho furnace of the Jones & Laughlin Co., Pittsburg, Pa., by a process patented by Joseph A. Shinn, of Pittsburg. The steam of molten slag is struck by two successive jets of water, and the product is said to be more compact and much stronger than slag sand produced by dropping molten slag into a pit filled with water, which is the ordinary process.

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ADVERTISING RATES: 20 *cents a line. "Want" and "For Sale" notices, special rates, see page XXIV-XXIX. New copy for standing advertisements should be received one week in advance of publication; new advertisements, Monday morning. The pages containing "Want," "For Sale" and "Proposal" advertisements are held open until Wednesday noon.*

We briefly recorded in our last issue the result of the referendum vote in New York on the thousand-ton barge canal project. As we then stated, the vote in the state generally was heavily adverse to the measure, the majority against it in various counties amounting to about 175,000. In Greater New York and in Buffalo, however, an enormous majority for the measure was polled, and the final result was a net majority in the entire state of over 250,000 for the barge canal.

The official canvass of the vote will be made sometime in December, after which, according to the statute, it will be the duty of the State Controller to sell state bonds to the amount of $10,000,000, the proceeds of which will then be available to begin work on the canal.

Since the result of the election was declared reports have been in circulation that the state officials at Albany would delay action for the present in the expectation that the legislature when it meets in January would repeal the barge canal bill. While there may be some small foundation for these rumors it is very doubtful whether any serious attempt to repeal the law will be made or if made whether it would be successful.

It is quite possible that a considerable proportion of the state officers believe that the barge canal project is a huge engineering mistake; and that they would be glad in the public interest to intervene to prevent its consummation. At the same time they would naturally not be averse to strengthening their party's hold on the country voters, who have been shown to be overwhelmingly against the canal.

On the other hand, it is difficult to find a pretext for the repeal. No argument against the barge canal project can be made to-day that could not equally well have been made a year ago, when the legislature passed the bill. It has been said that the new electric towing system, recently tested at Schenectady, might be made the excuse for repealing the bill; and it is not impossible that the matter may be brought up in the legislature; but this would seem a very doubtful reliance as an argument for the repeal of the law. Electric canal towing is not new, and while there can be little doubt as to its feasibility, it is doubtful how much it could accomplish to radically alter conditions on the existing canal.

Moreover, while the state officials may be sincerely desirous of saving the state from an unprofitable investment and of pleasing their constituents who oppose the canal scheme, they are very much alive to the political advantage resulting from the control of a hundred million dollar job.

Taking all these things into consideration, there seems little reason to doubt that the 1,000-ton barge canal work will go forward. This journal has opposed the project as a matter of public duty; and our editorial opinions have been supported by the ablest authorities in the engineering profession. Now that the work has been decided upon, however, it is proper for us to note that it will give employment for the next seven to ten years to a large number of engineers, and during the same time opportunities for business and profit to many contractors and manufactuers of machinery and engineering supplies.

We have not permitted this fact to influence in any way our discussion of the merits of the canal enterprise; but now that the taxpayers have voted to undertake the work, it is certainly gratifying to us that it will contribute in no small degree to the welfare of many of our readers and patrons.

Besides the force of inspectors, assistant engineers, resident engineers and division engineers, State Engineer Bond is given authority to appoint a special deputy, who will devote himself solely to the barge canal work. The Governor has also authority to appoint an "Advisory Commission" of five engineers, at compensation to be fixed by him, whose duties it will be to advise the State Engineer and the Superintendent of Public Works, when advice is requested.

The surveys made in 1900 will form a ground work on which final surveys preparatory to the letting of contracts can be made, and it is stated that a portion of the work can probably be put under contract by next spring.

In discussing contractors' guarantees on pavements in our last issue, Mr. S. Whinery expressed an opinion with which we are in hearty accord. He said:

> There seems to be no valid reason why, in contracting for well-known kinds of street pavements, the same procedure should not be followed that has been found most satisfactory in other branches of municipal public work. That is, to specify definitely and precisely the quality of the materials to be used and the manner in which the work shall be done, and then to enforce compliance by the contractor. The city would then know definitely what it was to get, and the contractor would be able to estimate with a fair degree of accuracy the cost of the work he is to do. *In other words, it would not then be necessary for him to combine in his business the function of the contractor, the prophet and the insurance company at the expense of the city.*

We have italicized the last sentence because it expresses with force exactly what we have frequently called attention to in these columns, namely, the economic shortsightedness of any practice that makes the contractor do an insurance business. If the engineer can not foretell the life of a structure, it is quite likely that the contractor himself cannot. A factor of safety has been called "a factor of ignorance," but with greater propriety may we term "a guarantee clause" a guarantee of ignorance on the part of somebody.

In the early days of asphalt and brick pavement construction, when engineers knew little as to the methods of production and manufacture of these paving materials, and nothing as to the prospective life of a brick or an asphalt wearing coat, there was justification for the use of a guarantee clause. But it seems to us time that methods of testing be developed, if they are not already in existence, that will enable the engineer to predict with greater accuracy than the contractor can predict what the prospective life of a pavement will be. Just as soon as guarantee clauses are abolished we shall see not only lower prices but better pavements; for careful tests will surely lead to improvements in the manufacture of paving materials.

Not long after the publication of one of our editorial comments on this subject of pavement guarantees, a marked newspaper was sent to this office, presumably as an argument against our advocacy of reform in pavement specifications. In this marked article was cited an asphalt pavement that had failed almost immediately after the expiration of a five-year guarantee; and it was seriously held that the ingenious but pernicious contractor had so contrived to select and to mix the asphaltic ingredients as to secure a failure like that of the wonderful one-hoss shay in the "Deacon's Masterpiece."

Let us assume, for the sake of argument, that certain asphalt contractors have so complete a mastery of the intricacies of asphalt pavement manufacture that they can tell to a day how long any given composition will last. Is this, then, a good argument in favor of guarantee? It seems to us rather that it shows that engineers should take one of these expert contractors gently but firmly to some secluded spot and "persuade" him to tell all he knows about asphalt. Even the "water cure" would be justifiably used as a means of such persuasion; for if certain asphalt contractors do know how to limit the life of their pavements with such a degree of nicety, then it is high time that their knowledge be rescued from oblivion before they pass to that bourne whence contractors never return.

It is always a source of gratification to the engineering profession when its work is publicly recognized and when on matters of large moment the public distinguishes between the engineer and the charlatan; between science and pseudo-science.

From this point of view, the action taken by the recent Levee Convention at New Orleans is a source for congratulation by every reputable member of the engineering profession. We print in another part of this issue the resolutions adopted by this convention in full; and a more complete and satisfactory recognition of the value of sound engineering it would be hard to find.

There is the more cause for congratulation in this action by the New Orleans convention, because the few opponents of the levee system have succeeded in securing an amount of publicity and attention wholly out of proportion to their numbers or to their standing and qualifications to render an opinion on a subject of much technical difficulty. On the other hand, the engineers who have defended the levee system have had to use, in meeting some of the specious statements of their opponents, arguments involving technical matters quite out of the comprehension of the layman. For illustration, we may take the oft-repeated statement of the anti-levee men that the bed of the Mississippi River is rising, and the plausible reasons which they have presented as to why this is taking place. To meet this argument, the Mississippi River engineers have cited actual measurements in proof of the falsity of the claim, but the method by which the measurements were made and their results analyzed, while clear enough to the engineer, is wholly beyond the depth of the layman.

We repeat, therefore, that under these circumstances, the emphatic acceptance by the New Orleans convention of sound engineering principles and their unqualified condemnation of quack schemes for Mississippi River control, is a cause for rejoicing by the engineering profession.

It is well known that a large part of the expense of garbage removal is due to long hauls by horses or mules, with the necessary driver in charge of each wagon. It is also well known that the procession of garbage wagons nearing and arriving at the place of final disposal during the greater part of the working day is often a source of annoyance to many people and in some instances a veritable nuisance. In the case of long hauls to distant disposal works removable wagon

bodies are sometimes transported by trains. There have been numerous proposals, from time to time, to haul garbage on electric railways. How many, if any, of these proposals have been carried out we cannot state, but elsewhere in this issue we illustrate and describe some garbage cars recently put in service by the Savannah Electric Railway Co. The contract price given in the article is certainly far below what the city could haul the garbage for by the usual methods, and the concentration of all the garbage upon the road during the early morning hours is certainly commendable. Only a few men and cars are required for this service, whereas ordinary haulage would require night work on the part of many men and teams. The method of disposal at the County Farm, so far as shown by the article and the view, is far from commendable in many respects, but this has little or no bearing on the merits of the system of transportation in use. Why should not this system be adopted by many other cities?

THE ISTHMIAN CANAL SITUATION.

We briefly recorded last week the outbreak of a revolution on the Isthmus of Panama on Nov. 3, the avowed object of which is to establish that region as an independent state, separate from the Republic of Colombia, and to treat with the United States for the construction of the Panama Canal.

It is an admitted fact that the revolution was planned and financed in New York city, and there seems extremely good reason to suppose that the interests behind the French Panama Co. were really at the bottom of the whole movement. Moreover, while it is doubtful whether any responsible United States official had any part in the planning, there is evidence that the State and Navy Departments were well aware of what was going on; and it is even said that herein lies the explanation of the failure to take up the Nicaragua route when the treaty with Colombia failed in September.

That the canal franchise and that only was the controlling reason for the Panama secession is too evident for proof. The revolution began on Nov. 3. By Nov. 7, M. Philippe Bunau-Varilla was in Washington equipped with credentials and full powers as Minister Plenipotentiary and Envoy Extraordinary of the Republic of Panama. It is not without interest to note that Bunau-Varilla is a brother of a former high official of the French Panama Co., that he is a citizen of France, and that he has been employed by the French company in capacities which may be judged from the following quotation from the Paris "Temps:"

> His conferences throughout the United States largely contributed to the final choice of the Panama route. He afterward went to Colombia and sought to induce the acceptance of the Hay-Herran treaty. This so strongly inclined popular sentiment at Panama in his favor that on the new republic being constituted and recognized by the United States it gave the post of honor to the man who had devoted all his energies to seeing the canal realized.

One is inclined to marvel at the extraordinary celerity of the "popular sentiment," which placed Bunau-Varilla in Washington within four days after the first outbreak of the revolution!

That the Government at Washington means to conclude a treaty with the infant republic at the earliest possible moment there appears every reason to believe. In fact, Washington dispatches of Nov. 8 stated that the outlines of a treaty had already been discussed in considerable detail by Secretary Hay and Bunau-Varilla, in advance even of his formal recognition of the latter by this Government. The State Department has also decided, it is said, that the Spooner act gives authority to conclude a treaty with the new republic since it assumes authority over the former territory of Colombia in which the canal lies. According to this, a new treaty when drafted will only have receive the approval of the Senate and the other body of Congress will have nothing to say in the matter. The task of securing the Senate approval to a new treaty is doubtless much ea ier than it would be to secure the passage of a new Isthmian canal bill by both houses of Congress.

It must be freely admitted that the United States is placed in an extremely difficult position. The general circumstances under which the revolution has occurred, the thinly-veiled participation of the French Panama Canal Co. as the chief actor in it, the fact that the whole movement has been engineered in the United States, and apparently with the knowledge of high Government officials, ought to make us exceedingly careful in our course, as we prize our nation's honor.

What is really the moving spirit in the revolution? Is it actually the people of Panama such as are fairly entitled to citizenship and a voice in public affairs, or is it the agents and employees of the French Panama Canal Co.? If the latter is the case, it is a very serious question whether a treaty should be made with such a government or whether the military and naval forces of the United States should be made use of to bolster it up. Suppose it were proposed to make with this state such a treaty as the Hay-Herran treaty with Colombia. Ought we to turn over $10,000,000 or indeed any large sum for the Panama franchise without knowing that something in the way of an orderly government is to receive the money and that it will not be merely turned into the pockets of the paid agents who appear to have engineered the revolution.

Again, if any scheme of joint control for the canal strip is to be exercised, we should know something of the government with which we are to assume partnership, to what degree it is truly representative, and what assurances can be found of its stability.

So far as foreign interference is concerned, it is doubtless true that we have a free hand. Europe is chiefly concerned to see an Isthmian canal built and will object to no measures that the United States may undertake with that end in view. Nevertheless, we shall not escape criticism if we make use of a revolution fostered for commercial ends to carry out our own commercial plans.

It is really questionable whether it would not be quite as honorable and more open and above board for us to assume entire control of the Isthmus without regard to the doubtful "Republic of Panama." The argument which is made for our interference at Panama—and it is admittedly a strong one—is that the end justifies the means. It would be undoubtedly vastly to the advantage of the people of Panama to have a strong and stable government in place of the series of chronic revolutions from which they have suffered; and it is to the advantage of the whole world to have a canal cut across the Isthmus without regard to the demands and pretensions of the petty republics who hold sovereignty over the land.

So far as this argument holds, it is clearly an argument for the radical rather than the half-way course; for the actual possession and government by the United States of Panama instead of making a bargain with an irresponsible group of filibusters, whose moral or legal authority to represent anything or anybody is exceedingly doubtful.

We could wish, as we have before stated, that for the sake of our national honor, the Government would cut loose from the whole Panama mess, permeated as it is with untold depths of graft and corruption, and undertake the construction of a canal at Nicaragua. Of this, however, there appears to be no prospect. In the days when the Maritime Canal Co. of Nicaragua was looking for a Government guarantee, Washington influences were all for Nicaragua and hardly a voice was raised concerning the Panama route except in condemnation. Now, however, the tables are exactly turned. Nobody has a dollar's interest in Nicaragua, while $40,000,000 of Uncle Sam's funds are to be paid on the final conclusion of the Panama franchise. Naturally, therefore, the sentiment at Washington is that the Panama route is the only route worth consideration at the present time.

SOME HINTS ON ESTIMATING COST OF RESERVOIR EMBANKMENTS.

A contractor whose experience has been largely in railroad earthwork, sometimes undertakes to do earth dam construction with which he is not experienced; and often he reads the specifications without a full realization of the import of the words. The rolling and the sprinkling specified are obviously going to add to the cost, but just how much the contractor often does not know and he "lumps it off," or in other words makes a guess. In our Construction News Supplement of Aug. 20 we commented upon the cost of picking pebbles out of a reservoir embankment, as is so often required in earth dam specifications. The railroad contractor seeing such a specification would hardly expect to be required to do something that seems so absurd to him as to gather up by hand every little pebble the size of a man's fist or larger and remove it from the embankment. Or, if he took the specification seriously, as he should, he might still delude himself with the thought that all pebbles could be removed at nominal cost with some sort of a rake drawn by horses. In such reasoning, however, there is great danger unless one has actually seen the horse-rake used successfully on the particular soil in question. Most materials suitable for earth dam construction contain clay, which, when dry, is also raked up with the pebbles, and when wet is even more certain to clog the horse-rake. Let this fact not be forgotten by the contractor who counts upon a nominal cost for removing pebbles.

We find it difficult to trace the origin of this requirement, but it is not improbable that this pebble removal specification was first applied to the back filling around pipes buried in the earth dam. There, it had some measure of justification, for it is well known that even concrete rammed against a form shows voids along the form unless special care is taken to spade back the stone, so as to leave clear mortar at the face. A specification drawn to cover a very small part of reservoir earthwork appears to have worked its way into general use for the embankment as a whole. Like the Irishman who found one pill so good that he forthwith swallowed the whole boxful, one often sees specifications originally drawn for use in small doses perverted to a wholesale application.

Passing from the "pebble picking" clause to the one requiring sprinkling "to the satisfaction of the engineer," it is wise for the contractor to carefully examine the earth that is to be sprinkled. A "dry" earth, one consisting largely of gravel, sand and loam, becomes quickly consolidated by sprinkling, and reduces the amount of rolling actually needed to make a very dense embankment. An earth containing much pure clay, however, if heavily watered becomes spongy and stalls the wagons even when "loaded light." Here again it is that the railroad contractor must be cautious in not underestimating the cost; for, aside from the expense of rolling an overwatered clay, there is the additional and greater expense due to decreased wagon loads, and to frequent miring of a team that may block an entire string of teams. An engineer of experience would not insist upon sprinkling to such an extent as to make a quagmire of an embankment, but we have seen inspectors of little experience use as a club "the sprinkling to the satisfaction of the engineer," and with this club force the contractor to a material loss in hauling and in rolling. Where a 10-ton steam roller is specified for rolling, the conditions are often worse. If one rear wheel of such a roller strikes an over-watered spot in a clayey embankment, there will be another period of lost time, not for the roller alone, but for the entire force, if the inspector is at all exacting. These are some of the factors that go to make reservoir earthwork more expensive than the contractor inexperienced in such work might expect.

LETTERS TO THE EDITOR.

Some Formulas for the Price of Sewer Trenching.

Sir: In your issue of Oct. 15 a correspondent advocates making a percentage increase in the price of trench excavation in case the depth of the sewer as laid exceeds the depth as shown by the plans, and that a like deduction is to be made for a decrease in depth, and he suggests that this percentage be 10% for each foot of depth of trench greater or less than that shown on the plans.

It can be shown both theoretically and empirically that the percentage must change as the depth of the trench increases. For illustration, let y equal the depth of the

trench, and x equal the cost of the sewer in cents per lin. foot, and w equal the width of the trench. An expression for the work of raising any infinitesimal depth or layer as dy may be expressed algebraically and will be proportional to the cost of the same.

From the sketch we have

$$y\ w\ dy = a\ dx; \qquad (1)$$

a = a constant.

Integrating the above equation we have

$$\frac{w}{2}\ y^2 = ax + b; \qquad (2)$$

b is another constant depending upon the cost of the pipe, cement, etc., or what we might call the fixed charges. From equation (2) we see that the cost of sewer varies with the square of the depth. Thus the extra labor involved in excavating a trench of depth $(y + e)$ over that of a depth of y is proportional to $2ey + e^2$ instead of proportional to e as would be the case in using a percentage.

I have shown practically the same fact empirically by taking the average of a large number of prices on sewers in which the depth of the trench was specified, and have derived the following formula for pipe sewers 6 to 20 ins. in diameter:

$$y^2 = \frac{(53 - S)\ x}{10} - 11\ S \qquad (3)$$

in which y and x are as above stated, and S is the diameter of the pipe. This would apply to average good excavation. I would suggest that a diagram constructed by the use of the above formula could be used to determine the increase or decrease in the price of the bid.

Those interested in formula No. 3 are referred to an article in the "Cost of Pipe Sewers" in the "Wisconsin Engineer," January, 1901, page 67. Very truly,

W. G. Kirchoffer, Consulting Engineer.

Baraboo, Wis., Oct. 28, 1903.

[We fear that our correspondent's demonstration of the cost of trenching by the use of the calculus is analogous to the use of a differential micrometer at the end of a hemp rope used in measuring the width of a fog bank.

Granting for the moment that our correspondent's assumption upon which equation (1) is based is correct, we note first that he has not integrated between the limits of 0 and the depth of the trench, as should have been done. Had he done this, the constant b would have been eliminated, and he would not have made the error of calling b "another constant depending upon the cost of the pipe, cement, etc." Referring back to his hypothesis in the sentence preceding equation (1), we see that nowhere in that hypothesis is there any mention of pipe or cement. How then, may we ask, can equation (2) contain an item not found in the hypothesis upon which the deduction is based? Obviously our correspondent's interpretation of equation (2) is at fault.

Going back to the hypothesis upon which the deduction rests, we find that it is not tenable; for the assumption is made that the cost of trenching is a function of the work of lifting the earth. This is not true, for, to begin with, it ignores the work of loosening the earth with picks, or otherwise. It likewise ignores the element of time involved in forcing the shovel into the earth before the work of lifting begins. It ignores the fact that in most deep trench work steam power is used for raising the buckets of earth, and it ignores several other important elements involved in practical work. Thus, up to depths of 5 or 6 ft. in certain earths no bracing is required, while for any deeper trenching the earth sides must be supported. Rehandling the earth after it has reached the surface is likewise not included in our correspondent's hypothesis. Backfilling and ramming are also ignored.

A formula to be of any practical value at all should include all these items and certain others, which equation (2) does not, since the hypothesis does not. But our correspondent maintains that his empirical formula, equation (3), based upon actual contract prices, is of the same form as equation (2). We are at loss to know how any empirical formula based upon the bids of numerous contractors working under varying conditions of soil, efficiency and price of labor, etc., etc., can be made to conform to any curve whatsoever. A somewhat extensive examination of contract prices of sewer trenching has served only to confirm our belief in their unreliability as guides to those not familiar with all local conditions. —Ed.]

Variations in the Size of Cement Barrels and in Specification Units—A Plea for Uniformity in Specifications.

Sir: A letter in your issue of Oct. 22 from Mr. Bruce C. Yates, inquiring as to the common practice of measuring cement, illustrates the necessity for greater exactness and uniformity of method in specifying the proportions of materials for Portland cement concrete. It is not sufficient, in my opinion, to simply state that the cement shall be measured "packed" or "loose," for there is a wide range in the meaning of these terms. By "packed" measurement may be meant the net volume occupied by the cement in the barrel, which in Portland cement I have known to vary from 112 to 125 lbs. per cu. ft., corresponding, respectively, to a barrel volume of 3.4 and 3.04 cu. ft.; or, as is more customary, the volume of the empty barrel may be selected as the unit. This barrel unit is further complicated by variations in the size of the barrel and in the method by which the volume is measured, whether between heads with the upper head removed (one way of measuring sand), or with both heads removed (another way of measuring sand). From a large number of measurements obtained by Mr. Frederick W. Taylor, M. E., and the writer, the average volume of a Portland cement barrel between the heads was found to be 3.5 cu. ft.; with both heads removed the average volume from end to end of staves is about 3.8 cu. ft.

From the above data it will be seen that a barrel of packed cement as defined by different engineers may vary from, say, 3.04 to 3.8 cu. ft., corresponding, respectively, to 125 and 100 lbs. per cu. ft.

The weight, on the other hand, of loose cement shoveled into a measuring box may vary from 83 to 106 lbs. per cu. ft., equivalent, respectively, to 4.6 and 3.6 cu. ft. per barrel. These figures represent extremes, but illustrate the differences of opinion which are liable to arise between engineers and contractors where simply "loose measurement" is specified. I have known engineers to differ in the selection of the unit of loose measurement from 3.8 to 4.12 cu ft. per barrel.

These figures are by no means theoretical. I have never known the smallest unit, 3.04 cu. ft., or the largest unit, 4.6 cu. ft., to be used, but I could cite the names of engineers, any one of whom is an authority on cement and concrete construction, who, in proportioning concrete, have adopted, respectively, barrel units of 3.1 cu. ft., 3.33 cu. ft., 3.5 cu. ft., 3.65 cu. ft., 5.8 cu. ft., 4.0 cu. ft., and 4.12 cu. ft. The standard net weight of a barrel of Portland cement is 380 lbs., hence these barrel units correspond to weights of cement varying from 123 lbs. per cu. ft. to 92 lbs. per cu. ft. Proportions 1:3:6 in the first case would mean 1 bbl., or 380 lbs., cement to 9.3 cu. ft. of sand and 18.6 cu. ft. of gravel; in the last case proportions 1:3:6 would stand for 1 bbl., or 380 lbs., cement to 12.36 cu. ft. of sand and 24.72 cu. ft. of gravel. In other words, concrete mixed 1:3:6 by one man may be called 1:4:8 by another.

It may be contended that this variation is of little moment so long as the unit is distinctly stated. This would be true were it not for the fact that it is customary in writing about or in discussing a piece of work to give the proportions of materials without stating the unit selected. Many of the records of tests of strength of concrete do not even specify the units used in proportioning the ingredients. It is especially confusing also to a contractor, who may find if he is not very careful in reading specifications that, say, 25% or 30% more cement than he had figured will be required to a cubic yard of concrete.

Take as an illustration of differing standards the specifications for the East Boston Tunnel which has just been completed, and the proposed Pennsylvania R. R. Tunnel. The former, in the construction contract for one of the sections, gives the proportions of cement and aggregate for the concrete masonry as 123 lbs. dry Portland cement to 2½ cu. ft. of sand and 4 cu. ft. of gravel, the sand and gravel "not to be packed more closely for the above measurement than is done by shoveling in a dry state into a measuring box." These proportions are verbally referred to as 1:2½:4. The Pennsylvania R. R. specifications* give as the proportions for most of their concrete work "1 part of cement to 2½ parts of sand and 5 parts of stone by volume," and define these proportions by stating that "one volume of cement shall be taken to mean 380 lbs. net, one volume of sand or broken stone shall be taken to mean 3⅓ cu. ft. packed or shaken down." If loose measurement of the sand and stone were employed, this unit volume would be increased to at least 3.8 cu. ft. On the face of it, the proportions for the two tunnels appear quite similar, a difference of only ½ part in the stone, but reducing the proportions of the East Boston Tunnel to a unit of 3.8 cu. ft. to the barrel (or 100 lbs. per cu. ft. of cement), we have approximately proportions 1:2:3¼ instead of 1:2½:4, a much richer mixture than the 1:2½:5 of the Pennsylvania R. R. I am not by this comparison criticising the proportions adopted in the two tunnels, for the conditions are entirely different and the design of the East Boston Tunnel naturally requires a richer mixture, but I am merely illustrating the effect of employing different units of measurement.

Thus far I have referred to specifications where the unit is distinctly stated. Cases are extremely frequent and are often alluded to in your correspondence columns of specifications where no statement whatever is made as to the unit of measurement or even as to whether the cement shall be considered packed or loose. In my personal files I have paving and sidewalk specifications (the former including concrete sub-pavement and the latter monolithic walks) of some sixteen of the largest cities in the country, all of them having over 90,000 population and covering a range of territory from Boston to Denver. The date upon most of these is 1898 or 1899. Mr. Yates' letter, above referred to, led me out of curiosity to look through these specifications to see how carefully the proportions were stated for the concrete work. I found that with only one exception the specifications of these sixteen cities fail to state the proportions by definite weight or volume, but give the materials simply in "parts," a few of them qualifying this by adding that the parts shall be "by measure" or "by exact measure."

As the selection of materials for concrete becomes more and more scientific, an increasing number of contracts will specify the exact weight of cement to definite volumes or weights of the aggregates, and eventually the present method of defining the proportions by parts may be abandoned. Until that time, and especially for the benefit of the smaller users of concrete, it seems to the writer that some authoritative unit of measure corresponding to a definite weight of cement, should be adopted for the sand and stone. At a recent meeting of the American Society of Civil Engineers it was suggested that a committee be appointed for investigating the properties of concrete. One of their aims may well be to fix definite standards of measuring proportions. A study of the subject leads me to favor either 3.5 cu. ft. or 3.8 cu. ft., corresponding to weights of 108 lbs. or 100 lbs. per cu. ft., and the balance appears to lie with the latter unit, viz., 3.8 cu. ft. per barrel or 100 lbs. per cu. ft. The reasons which may be suggested for advocating this are: (1) It is a fair average of the units most customarily employed, these generally ranging between 3.5 and 4.1 cu. ft.; (2) it is a convenient figure to remember—100 lbs. per cu. ft.; (3) it is approximately the volume of the average cement barrel with the two heads removed—a measure frequently used for sand, and (4) it is the unit employed in several cases by experimenters who have recorded their results in the Transactions of the American Society of Civil Engineers.

Sanford E. Thomson.

Newton Highlands, Mass., Nov. 5, 1903.

[Our correspondent is certainly right in urging the importance of definiteness in specifying how the cement in concrete is to be measured, and he is also right in urging the importance of uniformity in defining quantities in concrete specifications. This is a matter that the American Society of Civil Engineers may well act upon. It is not, however, a question that will be settled without considerable controversy, as there will be supporters of at least three different units of measurement, namely: (1) The packed barrel; (2) the loose volume; and, (3) the 100-lb. sack, or other weight unit. Since cement is now sold in sacks more often than in barrels, we question the expediency of mentioning the barrel at all in specifications. It will suffice simply to say that 100 lbs. of Portland cement shall be considered as being 1 cu. ft. Our correspondent regards this as being the best unit, and we are inclined to agree with him. Pure massive quartz weighs 165 lbs. per cu. ft.; hence, loose quartz sand containing 40% voids weighs practically 100 lbs. per cu. ft. Loose or slightly shaken Portland cement also averages about 100 lbs. per cu. ft.; hence, measuring by volume, thus, we secure a correspondence with the laboratory method of measuring by weight. This, we think, is an additional reason why it will be best to specify that measurement of concrete ingredients shall be by volume measure loose, and that 100 lbs. of Portland cement shall be considered as being 1 cu. ft. of loose cement.—Ed.]

*Engineering News, Oct. 15, 1903, p. 337.

Notes and Queries.

Mr. F. M. Turner, City Civil Engineer of Dayton, O., writes that Dayton is considering the establishment of a plant for repairing asphalt pavements and that he would like a list of cities operating such plants. We do not know of a city which has installed such a plant. Winnipeg, Man., has for several years operated a plant for constructing asphalt pavements (Eng. News, April 11, 1901) and an asphalt and tar macadam plant was also built at Geneva, N. Y., a few years ago (Eng. News, Nov. 20, 1902, p. 435). Repairs are made by the City of Chattanooga, Tenn., with a small portable asphalt heater, described briefly in Engineering News, of March 7, 1901, p. 162.

THE GARBAGE CARS OF THE SAVANNAH ELECTRIC RAILWAY COMPANY.

Under a recent contract the Savannah Electric Co. has provided six special cars for hauling all the garbage collected by the City of Savannah, Ga., to the County Farm, about four miles from the city. Each car has a capacity of 45 cu. yds. in bulk and 30,000 lbs. in weight. The cars are provided with hatch doors, A-shaped floors to facilitate dumping, and side dumping doors of the Hart type. The doors are controlled by a crank rod at the side and bottom of the car, arranged to operate all the doors at once. Rubber gaskets are provided to make the doors water-tight. The inclined floor is pierced with holes for draining the liquid matter in the garbage into a space beneath the A-floor. This inclosure is lined with galvanized iron, and three valves are provided for emptying the liquid. The valves are left open while the cars are being loaded, and closed just

EMPTYING A GARBAGE CAR OF THE SAVANNAH ELECTRIC RAILWAY CO.

before they are hauled away from the filling platform. Rubber gaskets are employed to make the dumping doors water-tight.

For loading, the platforms at the old garbage crematory have been fitted up, and the railway company has provided two side tracks for running its cars directly beneath the platform. The garbage is collected in dump wagons, which are emptied directly into the cars through the roof hatches already mentioned.

The cars are loaded during the day, but are hauled between 12 midnight and 6 a. m., when the passenger cars are off the line. At the County Farm the cars are run up a 2% grade onto a dumping trestle, where county convicts assist in dumping the contents and cleaning the cars, as shown by the accompanying view.

The cars were built by the South Baltimore Car Works. The contract with the city is for ten years, beginning in 1903, and the contract price is $20 a day on the basis of 75 tons (five cars) or less of garbage a day, with an extra charge of $4 per car for all cars over five a day. The service was started on Aug. 1, 1903.

After being dumped at the County Farm the tin cans, glass, rubber, metal, bones and other scrap is picked out of the garbage and the garbage proper is used for fertilizing the County Farm.

For the information given and view shown we are indebted to Mr. C. B. Kidder, Manager of the Savannah Electric Co.

CONVENTION OF THE INTERSTATE MISSISSIPPI RIVER IMPROVEMENT AND LEVEE ASSOCIATION AT NEW ORLEANS.

The most largely attended meeting ever held by this Association took place at New Orleans, La., on Oct. 28. Resolutions were adopted setting forth the confidence of the people of the Mississippi Valley in the levee system and urging larger National appropriations for its completion and maintenance. A committee of the Association will present these resolutions to Congress. Telegrams were read at the convention from the Governors of 20 states and from 125 members of Congress indorsing the objects of the Association and pledging support. We reprint the resolutions in full from the issue of the New Orleans "Picayune" of Oct. 29:

First—After years of actual observation and experience, and supported by the opinions of all engineers, whether from the Engineer Corps of the Army, or from civil life, who have been directly connected with the work of levee construction, we desire to affirm that we have the most absolute confidence in the sufficiency of levees when built according to correct standards, to protect the Mississippi Valley from overflow.

THE BED OF THE MISSISSIPPI RIVER IS NOT RISING.

In support of this declaration we beg leave to submit the following facts, which have been fully established: An elaborate and careful investigation, made under the direction of the Mississippi River Commission, wholly disproves the notion, which still prevails to a considerable extent, that the immediate effect of levee construction is to cause the bed of the Mississippi River to rise. If this were true, it would necessarily follow that the levees would need to be continuously strengthened and elevated, and thus all hope of protection would have to be abondoned.

In the years 1881, 1882 and 1883 an elaborate survey was made of the river bed from Cairo to the Passes, a distance of 1,063 miles. Four cross-sections to the mile were made, and 75 soundings were made to each line. The result of this survey was carefully plotted, recorded and preserved.

In the years 1894, 1895 and 1896, after the lapse of a period of thirteen years, a still more elaborate survey was made of that part of the river bed between the Arkansas River and Donaldsonville, La., a distance of 472 miles.

While local changes in the river bed are necessarily constantly happening by reason of the gradual movement down stream of the bends, and accompanying bars and pools, they of themselves signify nothing. Yet a comparison such as that which has been drawn from the result of the two extensive surveys mentioned would necessarily furnish proof that the bed of the river was rising if such were the truth. So far from the comparison indicating such result from levee construction, it was discovered that there is a general tendency to the establishment of a more uniform channel in depth and width and with greater capacity.

The comparison also brought to light the fact that the crests of the low-water bars, as well as those of the high-water bars, have been lowered.

If we turn to the evidence afforded by the records of the numerous gages established along the river, which have also been carefully recorded and preserved, we find that the low waters now are several feet lower than they were in the years preceding active levee construction, accompanied by an equal volume of water and an equal depth of channel. This unquestionably shows that the effect of levee construction has been to bring about a gradual depression of the river bed. This effect has been produced within the past few years, for prior to that time there was no such restraint of the flood waters as could leave any impress whatever, one way or the other, upon the river bed.

The notion that the bed of the river is rising has been somewhat revived since the flood of 1903, because of the fact that, at certain points, the gage reading showed not only unusually great elevation of the flood height, but irregular elevation. From this it has been deduced by some that at those places where the gage readings were the highest, there had been, as the result of levee construction, an unusual deposit of silt, thus raising the bed of the river. A simple explanation will destroy this theory:

In 1880, when the levees were by no means continuous and were altogether insufficient to affect the flood plane in any degree, the first thoughtful and scientific observation of the river began. This was because of the fact that the Mississippi River Commission then entered upon the discharge of its duties. It was noted that the rise and fall of the river was very different at different points. It was observed that the greater annual oscillations, which were of about 45 ft., were to be found at or near the mouths of the tributaries, such as the Ohio, the St. Francis; the Arkansas and the Red Rivers. It was also observed that the lesser annual oscillations, which were of about 25 ft., were to be found at intermediate points along the fronts of the great basins drained by these tributaries, as, for example, at Fulton, Memphis, Greenville, Lake Providence and St. Joseph.

A careful plotting of the gage readings at that time exhibited a smooth and regular high-water slope, but an exceedingly irregular low-water slope. This was caused by considerable depression of the river bed at or near the junction with the tributaries of the river, and a considerable elevation of the bed along the fronts of the great basins between them. For this reason it was noted that the rise in high water was much greater where the bed of the river was depressed at or near the points of junction with its tributaries.

It was observed that the discharge, at high water, at these points, because of these depressions, was something like 1,500,000 cu. ft. per scond, while along the intervening basin fronts the discharge was several hundred thousand feet less. This difference in discharge, ranging from a quarter to a half-million feet, was because of the escape of water over the river banks along these basin fronts. This escape of water undoubtedly caused the elevation of the bed along these fronts, which was noted, and we feel justified in affirming that when this escape shall have been permanently prevented by the construction of suitable levees, these elevated portions of the river bed will be gradually lowered to conform to the bed at the points of junction with tributaries, thus making a regular low-water slope. When this shall have been accomplished, undoubtedly the lowering of the river bed will steadily go on.

It has also been noted that during the flood of 1903 the heights attained by the flood in excess of those hitherto recorded were greatest at the points along these basin fronts, as, for instance, at Memphis, where the rise was three feet greater than any ever known.

The excess of flood height at the points of depression referred to was nothing like so extreme.

We, therefore, declare that in our judgment there is no warrant whatever for the assertion that the effect of levee construction has been, or will be, to raise the bed of the river, but, on the contrary, it is our definite conviction that the effect will be to cause a general and considerable lowering of the bed.

"RESERVOIR" AND "OUTLET" SCHEMES FOR MISSISSIPPI CONTROL CONDEMNED.

Second—We also desire to express our firm opposition to all schemes for reducing flood heights of the lower river by the construction of reservoirs or so-called outlets. We refer to and indorse fully all that is said upon this sub-

ject by the very careful and able report submitted in 1898 by the Commerce Committee of the United States Senate, which is so complete and elaborate as to exhaust the consideration of the question. We will add that all schemes which have ever been proposed for the relief of the river in times of flood by outlets or reservoirs would either prove wholly inefficient, or would cost such vast sums and require such constant care and expenditures as to entitle them to no consideration.

THE FLOOD OF 1903 A VINDICATION OF THE LEVEE SYSTEM.

Third—While the flood of 1903 was very nearly as great as that of 1897, and while the flood plane was greatly in excess of that of 1897, the protection afforded in 1903 over that of 1897 is so great as to satisfy the minds of all impartial investigators that so far as the test has gone the principle of protection by levee construction has been amply vindicated. In 1903 there were but six crevasses as against forty-three in 1897. With each recurring flood since levee construction began in earnest, the number of crevasses has grown smaller and smaller, and the protection afforded has grown greater and greater. As a result, investments of capital in all manner of industrial enterprises in the Mississippi Valley have increased until they are almost fabulous. The low-lying back lands which prior to that date were regarded as valueless, are fast being occupied and converted into homes for the benefit of our people. Towns and cities have sprung up in every direction. Railroads now traverse the valley so that nearly every part of it is reached by them. All of this affords evidence of the strongest possible conviction on the part of the people that the time is sure to come when they will have absolute protection from the floods of the river.

Theorists may argue against the efficiency of levees, but they do so in vain. The strong common sense of the people responds by rejecting their theories. The work must go on. It cannot now stop. Too much money has been invested in levees to suffer them to be destroyed, and unless they are prosecuted to completion they will be destroyed. The enormous investments made because of them and in reliance upon their completion cannot in good faith be abandoned now to the devastation of the floods. We presume that no man can be found at this stage of the work to suggest that the plan of protection by levees should be abandoned, at least until a full and complete test has shown them to be impracticable.

Fourth—The following abstract of the report of the Mississippi River Commission just made and hardly yet published, gives the very latest opinion of the Commission upon the levee question, and is so comprehensive and pertinent that we give it at length, as follows:

The past flood established, more clearly than has any previous one, both the importance and the practicability of a complete and sufficient levee system. In its present condition, incomplete both as regards extension and dimensions, it gave substantial protection to three-quarters of the alluvial valley, and its interests, which under equal floods conditions without levees, would have been a lake from 20 to 80 miles wide from Cairo to the gulf. This improvement made during the past six years has reduced the number of crevasses between Cairo and New Orleans from 35 to 6. Of the area overflowed this year, five-eighths was the direct result of back water from the lower ends of the basins and overflow through unbuilt parts of projected lines, and only three-eighths from breaks in the levees, notwithstanding their unfinished condition as regards both grade and section.

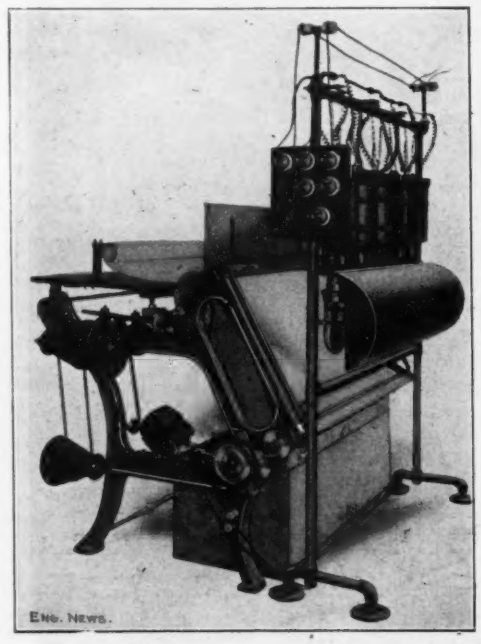
Under these circumstances the importance of the earliest practicable completion of the work is apparent. If the flood damages of 1903 may be approximately estimated at five millions of dollars, the previous expenditure of that sum in permanent work would have largely, if not entirely, prevented them. Every year's delay in completion incurs the risk of similar loss. When the system shall have been completed, the cost will have been increased by many millions of dollars, and the development of the valley delayed by many years of anxiety and disaster, which coud have been saved by continuous work on a scale commensurate with the importance and magnitude of the improvement. The state levee districts realize this. Most of them have anticipated their revenues as far as practicable, and several have now under consideration plans for such increase of resources applicable to the work as will shorten the time of completion. The Commission is so impressed with this view of the subject that it considers it for the best interest of the work to now make contracts for levee construction to the extent of two million dollars, as provided for in the River and Harbor Act of June 13, 1902, from the amounts to be appropriated for the fiscal years ending June 30, 1905, and June 30, 1906. Furthermore, it suggests that if Congress should think proper to make additional provisions for levee construction during the fiscal years ending June 30, 1905, and June 30, 1906, the sum of two million dollars in addition to the amounts already provided can be judiciously and advantageously expended during each year.

TRANSPORTATION ON THE MISSISSIPPI RIVER.

Fifth—In addition to the protection of the lands of the Mississippi Valley from the floods, it is a matter of supreme importance that the mind of the nation should be kept constantly advised of the commercial importance of the Mississippi River as a highway of commerce. The marvelous growth of railroad building within the last quarter of a century has so diverted the attention of the public from the Mississippi River as a means of transportation that it has been to some extent lost sight of. It has remained, however, a constant safeguard against undue rates of transportation and promises, in the near future, to become once more as active a factor in interstate commerce as it ever has been in the past. This is owing, first, to the almost unparalleled increase in industrial activity throughout the valley; and, second, to the demonstration which has been made in recent years, that, by means of hydraulic dredges a sufficient channel for low water navigation can be secured and maintained. We earnestly express the hope that the work of the Mississippi River Commission in this direction be pressed as rapidly as can be properly done, with a view to opening up the great river once more so that the people may fully enjoy the extraordinary facilities which it is capable of supplying for the cheap and steady exchange of their commodities. Levee construction is, undoubtedly, essential, even if all thought of reclaiming the fertile lands of the valley should be abandoned, for, without levees, all river commerce during periods of overflow would necessarily cease.

NATIONAL CONTROL OF MISSISSIPPI RIVER WORKS.

Sixth—The work of levee construction has been carried on by the co-operation of the United States government through the agency of the Mississippi River Commission, with the levee organizations of the several riparian states. Of the amount expended in this work, the government has contributed, in round figures, about one-third. The people have subjected themselves to such heavy taxation in furnishing their contributions until they have already overburdened their resources in this regard. It is the opinion of the residents of the great valley that the difficulties and magnitude of the work and the vast benefits to result from it, are such that, in common justice, the burden should be placed upon the strong shoulders of the Federal government, and that the work should be urged to speedy completion. By suitable annual appropriations, this can soon be accomplished, thus securing not only safety, but great economy. Therefore:



An Automatic Continuous Feed Electric Blue-Printing Machine.

Resolved, That, in the judgment of this convention, the protection of the Mississippi Valley from floods is of such national importance as not only to justify, but to make it the duty of the general government to undertake it and press it to the speediest possible completion. If, for any reason, the exercise of sole jurisdiction at this time by the general government, should not be deemed advisable, then this convention urges most earnestly that Congress make, at its approaching session, such appropriations as are recommended by the Mississippi River Commission in its recent report.

Resolved, further, That the system of river improvements in the Valley of the Mississippi from its head waters to the gulf and in the Valley of the Ohio, and other tributaries, now provided for and those which may hereafter be provided for by Congress under the supervision of the United States Engineers, meets our hearty commendation, and should be prosecuted to completion without unnecessary delay.

Resolved, That the attention of Congress is invited to the serious disasters which have befallen those residing at or near St. Louis, Kansas City, and other localitites by reason of the recent great floods, and the Secretary of War is respectfully requested to cause an inquiry to be made with a view to the preparation of suitable plans for the prevention of a recurrence of such injuries.

Be it resolved, That the convention of delegates representing the states of the great Mississippi Valley, from Duluth to the Gulf of Mexico, gives its unqualified approval to the movement for the construction of a waterway connecting the Great Lakes at the north with the Mississippi River and the Gulf of Mexico at the south.

We recognize the expenditure of $35,000,000 by the Sanitary District of Chicago as a practical demonstration in the furtherance of this project. We express the hope that the Senators and Representatives in Congress from the various states represented in this convention will give their encouragement and assistance to congressional legislation in favor of the completion of the deep waterway, to which the Mississippi Valley states have already given their approval, and to which the state of Illinois and the Sanitary District of Chicago are committed as a matter of policy, and by great financial expenditures already made.

AN AUTOMATIC CONTINUOUS FEED ELECTRIC BLUE-PRINTING MACHINE.

We illustrate in the accompanying engraving a machine for making automatically and continuously blue prints from tracings of any size from a few inches square to several yards in length, and a yard or more in width, depending upon the gage of the machines employed. The printing is done by electric light so that the operation of the machine is independent of weather conditions. In addition, the machine is claimed to have the following advantages over other electric blue-printing devices. The printing is continuous; the contact is perfect and automatic, and is entirely independent of the skill or care of the operator; there are no curtains, springs or levers to require adjustment; the machine is operated by an electric motor, leaving the operator free to attend to other duties.

The illustration in connection with the following description will explain the construction and operation of the machine quite fully. The portion of the machine by which the printing is accomplished consists of two continuous bands of transparent celluloid. Each of these bands is held firmly in position by two rollers, one of which is an idler and the other a driving roller which actuates the band. The driving rollers are geared together and revolve at exactly the same rate of speed, causing the bands to move synchronously. The transparent celluloid bands, as they travel from one roller to the other, pass over a convex surface or "hump," shaped in the section of a cylinder, and are held in absolute contact with this surface and with one another by an automatic adjustable tension device. In order to make a print the tracing and sensitized paper are introduced between the transparent bands, by means of which they are held in absolute contact with one another, and in this position pass in front of the electric lamps; the printing is thus accomplished automatically and the action of the machine being continuous, prints of any length can be made, or one separate print after another may be obtained in rapid succession.

The lamps and motor are furnished for the direct or alternating currents of 104, 110 or 220 volts. Either four or five enclosed arc electric lamps are used. These lamps are each arranged to pass from 2¾ amperes on the 220-volt current to 5 amperes on the 104 and 110-volt currents, making a very low cost of operation. The motor is 1-10 HP., and can be furnished for any current or voltage. By a cone device the speed of the bands may be altered so that the tracing and sensitized paper pass at a rapid or slow speed across the convex surface, thus making either a long or short exposure.

Prints can be made at a maximum speed of one minute exposure, or a minimum speed of 10 minutes. This great range permits all kinds of sensitized paper to be used. Tracings of any density, including paper negatives, drawings on bond paper or crumpled tracings are all printed with equal ease. Referring to the illustration, it will be seen that the prints fall into an inclosed dark box, whence they can be removed as desired for washing and drying.

The device illustrated is made by Williams, Brown & Earle, of Philadelphia, Pa., to whom we are indebted for the information from which this description has been prepared. According to the published circulars of this company, a complete machine for printing tracings 30 ins. wide will cost $400.

THE PROTECTION OF STEEL IN BALLASTED FLOOR BRIDGES.

In bridges having a solid floor construction, a thorough protection of the metal work of the floor is highly important. This is especially the case where the floor is covered with ballast, as the metal is exposed to mechanical injury from the scratching of the stone, as well as to corrosion by damp or moisture. Where a bare floor is used, as in some railway bridges over city streets (Eng. News, Feb. 22, 1900), a good paint will perhaps be sufficient protection, but something more than this may be necessary or economical for a ballasted floor, and the use of asphalt paint and asphaltic concrete has proved very satisfactory in many cases. For solid floors the use of steel throughout is the common practice, but some roads prefer to use a decking of creosoted plank upon the steel framing of the floor system. It is to be noted that the trough type of solid floor seems to be losing favor, preference being given to the simpler construction of transverse I-beams supporting a floor or deck of steel plate or creosoted planking. We have recently undertaken an inquiry as to the practice of different railways in regard to solid-floor construction and present herewith some of the results of our investigations. Other notes on this matter will be found in the articles on the Chicago railway track elevation work in our issues of Jan. 11, and Feb. 22, 1900.

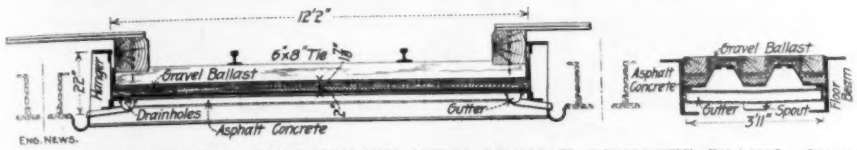


FIG. 1. TROUGH FLOOR CONSTRUCTION, WITH ASPHALT CONCRETE FILLING; CHICAGO & NORTHWESTERN RY. BRIDGE OVER MAIN ST., ROCKFORD, ILL.

E. C. Carter, M. Am. Soc. C. E., Chief Engineer. W. H. Finley, M. West. Soc. C. E., Principal Assistant Engineer.

CHICAGO & NORTHWESTERN RY.

At a meeting of the Western Society of Engineers in November, 1901, Mr. W. H. Finley, Principal Assistant Engineer of the Chicago & Northwestern Ry., described a method used by him, and we quote the following extract from his remarks:

About eight years ago the railway company I am with had occasion to put in a bridge at Rockford, Ill. The city insisted that the floor be made water-tight, and I decided to put in a solid ballasted floor, and to protect it by covering the troughs with asphalt mixed with sand in proportion of about 1 to 1. Recently it was necessary to renew the ties and the gravel ballast that was put on top of this asphalt, and we took that occasion to make an examination of the asphalt covering. The asphalt concrete was about 2 ins. at the bottom of the troughs. The California asphalt used did not contain coal tar or any of its products, and it was found upon examination that there was considerable life and elasticity to the concrete. In fact, it was almost as good as the day it was put in, and the trough from which it was taken was absolutely as dry and well preserved as on the day it was put in. We have since built a number of ballasted floor bridges, using pure asphalt only, mixed with sand, and I am satisfied that it is entirely practicable to protect an ordinary ballasted floor bridge so that there will be no deterioration of the iron or steel. This particular bridge has been absolutely water-tight during all these years. It is situated over Main St., in Rockford, Ill., and if there was any leakage we would have heard of it very promptly.

I believe all the asphalt compositions used in Chicago contain either coal tar or its products, and in cold weather the concrete cracks and permits the entrance of water between the covering and the iron, with a consequent rapid deterioration of the iron.

I have found in my experience in applying asphalt to metal bridges that it was absolutely necessary to have the iron clean as far as grease and dirt are concerned, and that in no case should the iron be damp, cold or chilled. If hot asphalt is put on iron, wood or stone where there is any moisture, it will peel off and will not adhere, so that in all cases where it has been applied in cold weather, we have had to heat sand and spread it over the iron work to take out the chill. In summer time the sun is usually hot enough to heat the iron, and artificial heat is not necessary. In one case where we had to take up a subway covering that leaked, we found that it had been applied without first drying off the stone. After the covering was removed, the stone and metal work was cleaned and heated, and a new coating of asphalt put on. We have had no trouble since and it is absolutely water-tight.

The cost is not excessive, and of course after you once make up your mind as to whether you will put in a solid floor bridge or an open floor bridge, the extra cost of this protection is a small amount compared with the total cost.

The Chicago & Northwestern Ry. now uses asphalt paint and asphalt concrete for its solid floor bridges, which are of varying types. The directions specify that all metal work exposed to the ballast must be treated in the following manner. If the surface to be protected has not been given a shop coat of asphalt paint, the metal must be thoroughly cleaned of all oil, paint, rust and dirt. The surface must be perfectly dry before the asphalt is applied, and if the temperature is low, the surface must be heated by covering it with hot sand, which is swept off in advance of the asphalt.

It is only necessary to heat the steel floor in the field sufficiently to take the chill out of the metal. It is better to heat the metal in all cases if the temperature of the atmosphere is not high enough to do this. The surface thus treated is first coated with a painting coat of the hot asphalt, and then with a layer not less than 1 in. thick at any point of hot asphalt and heated torpedo sand, mixed in the proportions of 1 of asphalt to 4 of sand. This layer is spread out and put in place with hot tamping irons, such as are used in asphalt paving. A final coat of hot asphalt about ¼-in. thick is then spread over the surface, sprinkled with sand, and thoroughly ironed over with smoothing irons, also similar to those used on asphalt paving. The thickness of the asphalt and the slope for drainage conforms to the drawings of the several bridges. The asphalt should be heated to not less than 350° F., and the sand should also be heated. The asphaltic concrete should be mixed thoroughly in the kettle before being applied.

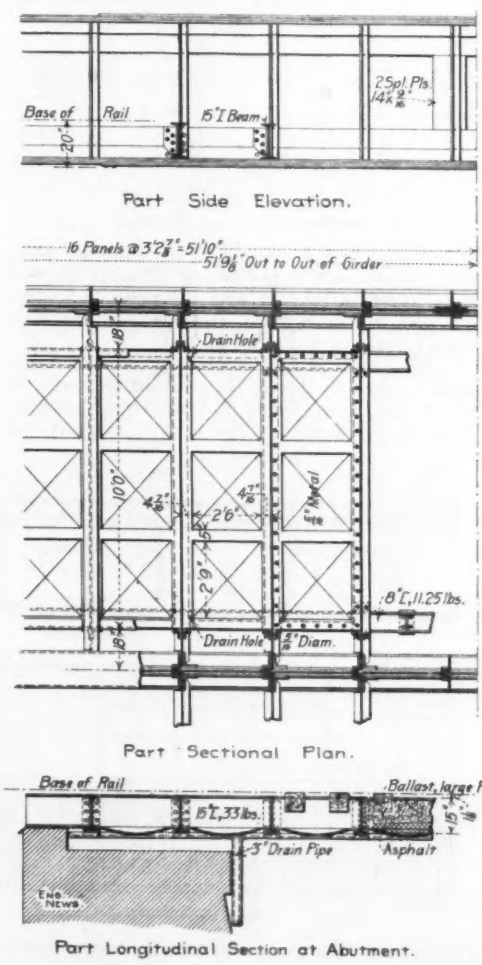


FIG. 2. BUCKLE PLATE FLOOR WITH ASPHALT CONCRETE FILLING; CHICAGO & NORTHWESTERN RY.

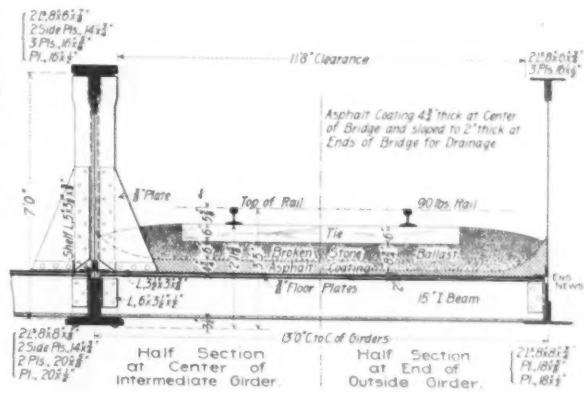
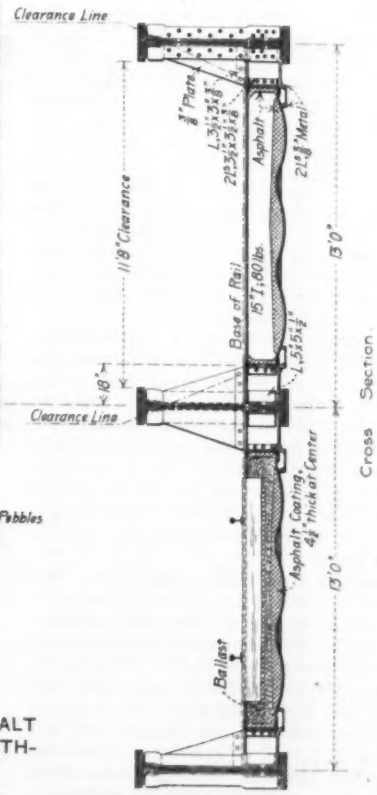

118" Clearance
Asphalt Coating 4½" thick at Center of Bridge and sloped to 2" thick at Ends of Bridge for Drainage
⅜" Plate
Top of Rail
90 lbs. Rail
Tie
Broken Stone Ballast
Asphalt Coating
⅜" Floor Plates
15"I Beam
130" C to C of Girders
Half Section at Center of Intermediate Girder.
Half Section at End of Outside Girder.
ENG. NEWS.

FIG. 5. RECENT DESIGN FOR SOLID FLOOR BRIDGES WITH ASPHALT CONCRETE COVERING, FOR TRACK ELEVATION WORK; CHICAGO & NORTHWESTERN RY.

More than a hundred solid-floor bridges have been protected by the asphalt concrete, including the bridges on the company's Chicago track elevation work. The process is being used for additional bridges, and it is proposed to apply it to all structures where a ballasted floor is used. Three single-track spans have had this covering in place about 11 years, and it is still in perfect condition. The asphaltic concrete does not deteriorate, neither does it separate from the steel if it is properly applied in the first place. After eight years, portions have been examined and removed, and it was adhering so well to the steel that the mill scale was taken off with it, leaving the metal bright underneath. Several grades of asphalt have been used: California, Utah, and Lake asphalt (both Trinidad and Bermudez); the Cimmerian brand of asphalt paint is now being used.



It is very necessary that the surface to be treated should be not only perfectly dry, but also warm. If done in cold weather, the surface should be heated by applying hot sand, which is swept back as the asphalt is spread. If the surface was treated with the sand blast this would put it in an ideal condition, but this plan has not been resorted to. Formerly it was required that the surface to be asphalted should not be painted or otherwise treated at the shops, but it was not always possible to hold the shop painters in check. The bare metal rusted very rapidly in the field, but in the future a shop coat of asphalt paint will be applied to the surfaces to be protected by asphaltic concrete, these surfaces being thoroughly cleaned before the application of the paint. The thickness of the concrete varies, but 1 in. will give sufficient protection. The cost varies from 10 to 15 cts. per sq. ft. The engineers consider that the process is not only so satisfactory as to warrant this expense, but that it is the only method used as yet that will give an absolutely water-tight floor and protect the metal from corrosion. With one or two exceptions, the work has been done in all cases by the company's forces.

As already noted, the Chicago & Northwestern Ry. has different styles of solid floor construction in service. Fig. 1 shows the trough floor of the bridge already referred to as crossing Main St. at Rockford, Ill., on the Galena Division. The troughs are partly filled with asphalt concrete, the thickness of which is about 2 ins. at the middle and tapers off to each side to form a slope to the drainage holes. Gravel ballast is filled in and over the troughs, and in this gravel the ties are embedded, as shown.

The buckle-plate floor of the bridge over Vine Ave., at Highland Park, Ill. (on the Wisconsin Division), is shown in Fig. 2. This is a through, plate-girder bridge with transverse 15-in. I-beams, resting upon the lower chord angles. The Carnegie buckle-plates are of 5-16-in. steel, and their flanges rest on the bottom flanges of the I-beams. These plates are laid with the concave side upward, forming a series of pockets. These are filled with asphalt concrete, to a maximum depth of 4½ ins. at the middle pocket. Above this is filled in screened gravel or broken stone ballast to the level of the tops of the I-beams, the ties being embedded in this and being 1½ ins. clear above the beams. Large pebbles are piled over the drainage nipples to prevent them from clogging.

A deck-plate floor as used for bridges over some of the streets crossed by the track elevation work on the Mayfair cut-off (Wisconsin Division) is shown in Fig. 3. These bridges are through plate-girder structures, with transverse 15-in. I-beams fitted between the bottom chord angles and shelf angles riveted to the girder webs. The I-beams support a flat deck of ⅝-in. plates. This floor is covered with 2 ins. of asphalt concrete, and 15 ins. of ballast. The girders span the roadway only, and the sidewalk spans consist of eight lines of 15-in. I-beams, covered with the same deck plating and having a facia girder riveted to each outer I-beam to retain the ballast.

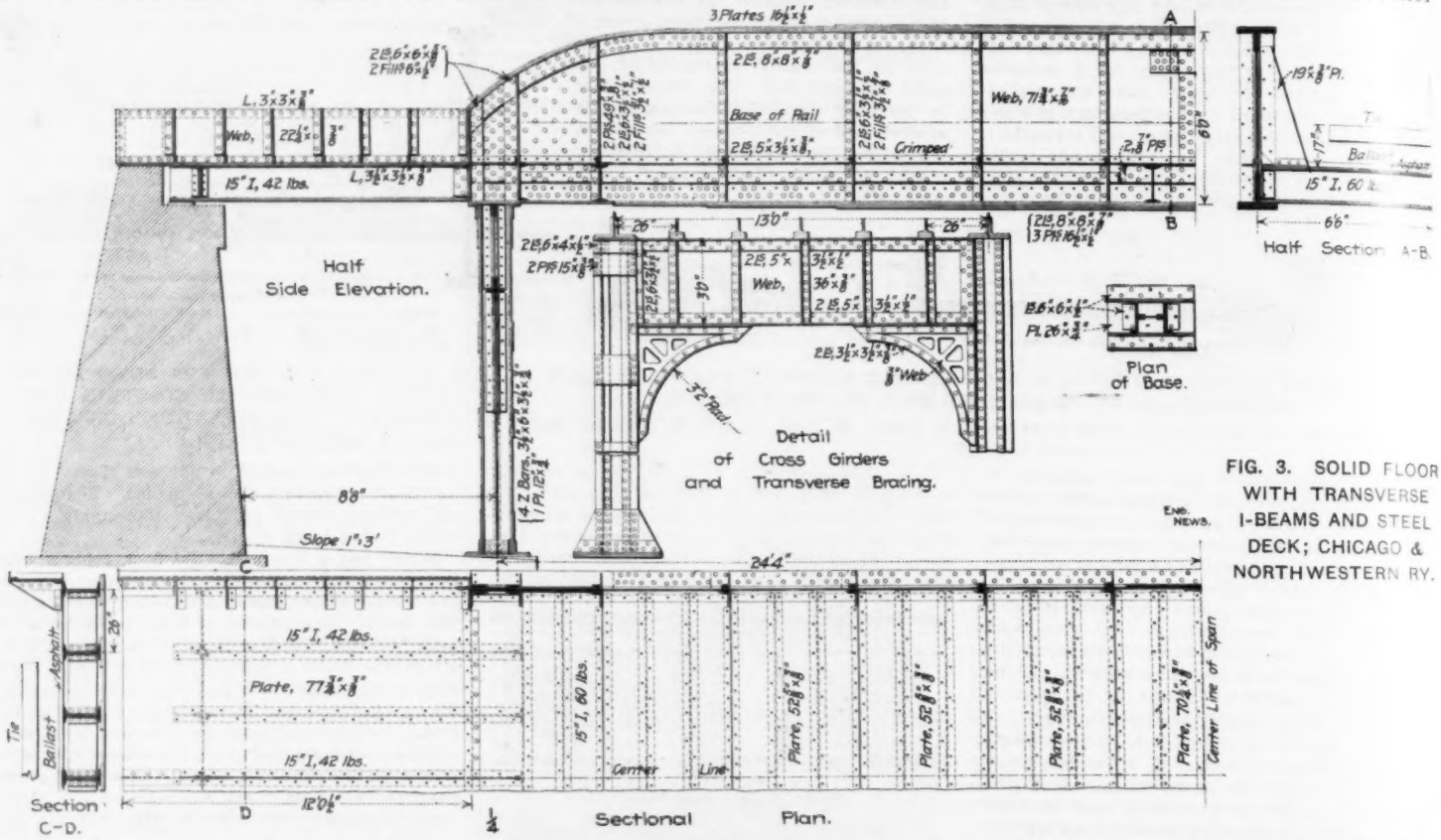


FIG. 3. SOLID FLOOR WITH TRANSVERSE I-BEAMS AND STEEL DECK; CHICAGO & NORTHWESTERN RY.

Another form of deck-plate floor, used for some other bridges on the Mayfair cut-off, is shown in Fig. 4. Here there are 20-in. longitudinal I-beams, 13 in number, spaced 26 ins. c. to c., and carrying a ⅜-in. steel deck plate with facia girders as noted above.

In Fig. 5 is shown a recent design for girders and floors for bridges on track elevation work in Chicago. This design provides for a flat deck floor on transverse I-beams. The asphalt coating is 4¾ ins. thick at the middle of the span and slopes to a thickness of 2 ins. at the ends to provide for drainage. At each side the asphalt is carried up against the girder webs so as to protect them from the ballast. The following is an estimate of the cost of the superstructure of four-track, five-girder, ballasted floor bridges of 66-ft. clear span, of the type shown in Fig. 5.

MICHIGAN CENTRAL R. R.

On the Michigan Central R. R., a coating of Assyrian asphalt has been used quite extensively as a protection for the floor plate and side angles on deck bridges with ballasted floor. It has been found to answer the purpose very well. The asphalt is applied hot in a liquid state, and poured over the top plate or deck plate to a thickness of about ⅛-in., the plate being heated before the asphalt is poured upon it. Fine gravel is then sprinkled over it, and upon the floor is laid crushed stone ballast 12 ins. deep. The heating of the metal floor was originally effected by building wood fires on platforms underneath the bridge. More recently, however, the floor has been heated from the top by means of special gasoline torches, and this has been found more satisfactory. The temperature to which the metal is heated is approximately 120° F., or so hot that the bare hand cannot be held upon it for any length of time.

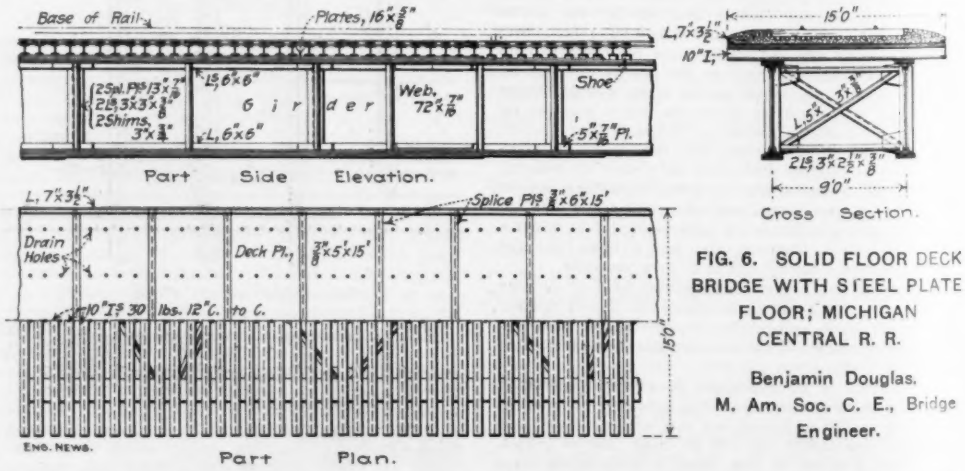


FIG. 6. SOLID FLOOR DECK BRIDGE WITH STEEL PLATE FLOOR; MICHIGAN CENTRAL R. R.

Benjamin Douglas.
M. Am. Soc. C. E., Bridge Engineer.

Estimated Cost of Ballasted Floor Bridges; Chicago & Northwestern Ry.

Size.	Beams: Wt. per ft., lbs.	Beams: Spacing, ins.	Cost: Floor: Per sq. ft.	Cost: Floor: Total.	Cost: Girders.	Cost: Total.
15 ins.....	...	...	$1.92	$7,089	$8,556	$15,645
15 "	42	15	1.97	7,273	8,556	15,829
15 "	65	20	2.19	8,100	8,556	16,656
15 "	100	24	2.55	9,400	8,556	17,956

A single-track deck plate-girder bridge with ballasted floor is shown in Fig. 6. The girders are 9 ft. c. to c., and across the top chords are laid 10-in., 30-lb. I-beams, the beams being 15 ft. long and spaced 12 ins. c. to c. Shims or shoes are used to level up the beams for the cover plates. Upon the beams is riveted a deck or floor of ⅝-in. steel plates, 5 × 15 ft., with ⅝-in. splice plates 6 ins. wide and 15 ft. long at the joints. Along each edge of the floor is riveted a steel angle 3½ × 7 ins., 15 lbs. per ft., having its 7-in. leg vertical. This is raised from the plate by a ½-in. shim over each I-beam. The weight of this floor system is about 160 lbs. per sq. ft. This construction is found to be very satisfactory and economical. For much of the above information we are indebted to Mr. Benjamin Douglas, Bridge Engineer of the Michigan Central R. R.

NEW YORK CENTRAL RY.

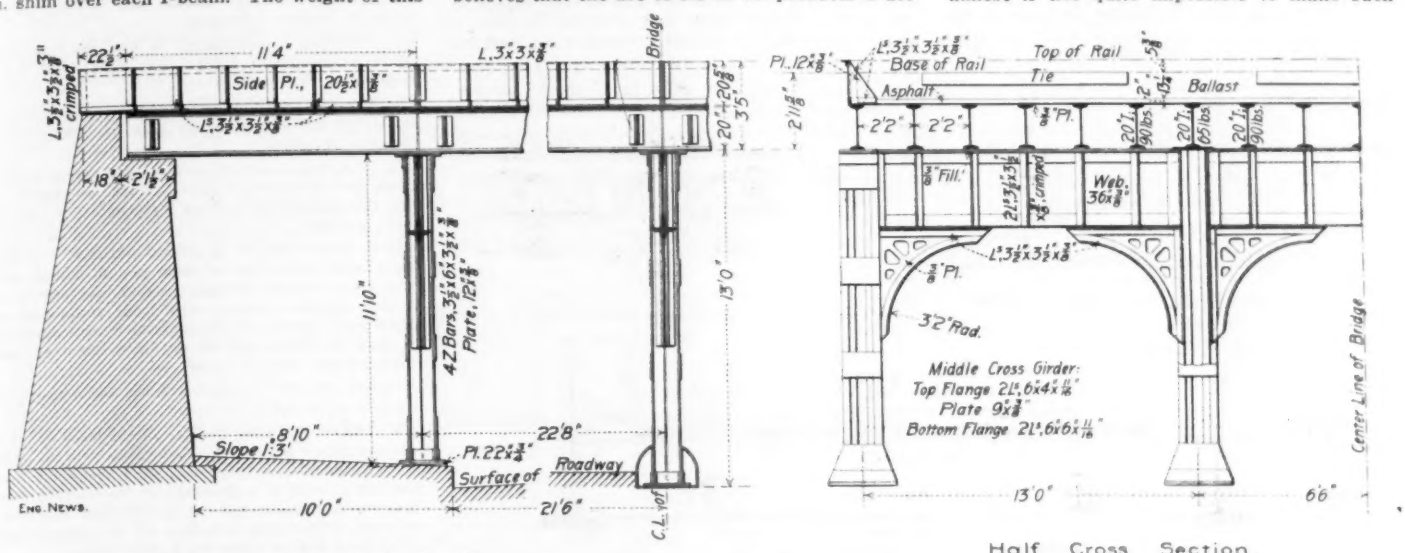
On this railway two styles of solid floor bridges are used: (1) I-beam and plate construction, which consists generally of rolled I-beams placed quite close together, and covered with a deck plate on top, this plate being generally 7-16-in. thick; (2) Trough floor construction, consisting of vertical web plates and horizontal top and bottom plates connected together with angle irons. Mr. Olaf Hoff, Engineer of Structures, states that the first plan is now the preferred construction.

All solid steel floors, like the rest of the steel work, receive first one priming coat of red lead paint. The underside of all solid floors receives, after erection, two coats of New York Central Ry. asphaltum varnish, the same as all other structural work. The ballast side, however, of solid floors, instead of being painted with this varnish, receives a coat consisting of 10 parts (by weight) of refined Trinidad asphalt, and 30 parts of straight run coal tar pitch, well mixed and heated to a temperature of 300° F. The ballast side of the solid floor then receives a so-called binder, which is an asphaltic concrete, composed as follows: Clean sharp gravel screened to ¼-in. size, and No. 4 asphalt paving composition, in proportion of 1 cu. yd. of gravel to 1½ gallons of composition (that is, of sufficient quantity to fill the voids). The gravel and the composition are first heated to a temperature of 300° F., and then thoroughly mixed together at that temperature. The mixture is then applied to the solid floors so as to vary in thickness from 1½ to 3 ins., affording drainage to the low points, where drainage nipples are placed.

In the troughs, this asphaltic concrete is rounded up against the sides of the troughs, but not carried up along the sides. When the material has hardened, the floors are ready to receive the ballast. In trough floors, the ties are generally placed in the troughs, the ballast filling up the space between the tie and bottom and sides of trough. It will be noticed that a mixture of tar and asphalt is used by Mr. Hoff, while it has been stated above that Mr. Finley (Chicago & Northwestern Ry.) believes that the use of tar or tar products is detrimental and that only pure asphalt should be used.

ILLINOIS CENTRAL R. R.

For the trough floor bridges on the Chicago track elevation work of the Illinois Central R. R. at Hyde Park, some of the tracks have been strengthened by riveting deck beams underneath the trough plates. This is done with the expectation of putting asphaltic concrete into the troughs, filling the floor about 2 ins. above the level of the trough with this composition, and then placing gravel or crushed stone ballast upon this. The concrete filling has not yet been applied. On the St. Charles Air Line track elevation, in Chicago, the steel floor plates were covered with a cement composition made by the M. W. Powell Co., under a guarantee for a period of years. This is covered with fine gravel and is in the nature of a roofing composition. This work was all done under the direction of Mr. H. W. Parkhurst, Engineer of Bridges and Buildings, of the Illinois Central R. R.



Part Longitudinal Section. Half Cross Section.

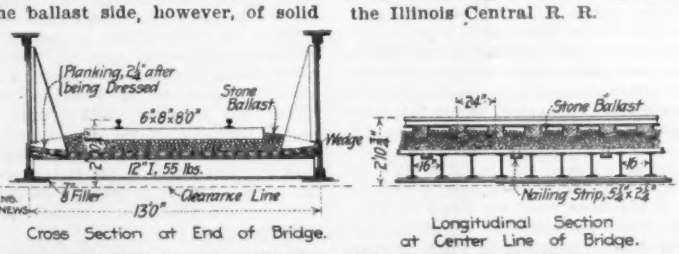
FIG. 4. SOLID FLOOR WITH LONGITUDINAL I-BEAMS AND STEEL DECK; CHICAGO & NORTHWESTERN RY.

CHICAGO & ALTON RY. AND ATCHISON, TOPEKA & SANTA FE RY.

For the ballasted floor bridges on the joint track elevation from 18th St. to Ashland Ave., Chicago, a solid timber deck or floor is used instead of a steel deck. This work is being done jointly for the Atchison, Topeka & Santa Fe Ry., the Chicago, Madison & Northern Ry., and the Chicago & Alton Ry. It is under the direction of Mr. G. W. Vaughn, Engineer in Charge. The construction is shown in Fig. 8.

Between the plate girders are placed 12-in I-beams spaced about 16 ins. c. to c. On these is laid a floor of creosoted pine planks 3 × 6 ins., carrying 12 lbs. of dead oil per cu. ft. of timber. At first tongued and grooved planks were used, but in the later floors there is simply a calking joint, as used on a ship's deck. The planks are laid close and are held by nailing strips below, as shown in Fig. 7. The seams are then calked and pitched, rendering the floor water-tight. The floor so laid and calked is covered with a coating of hot coal tar, and over this while hot is spread a light coat of sand. When cold a similar coating of tar and sand is spread.

No steel plate decks have been used in connection with the ballasted floor on this work, it being considered, as the result of observation, that it is almost if not quite impossible to make such a floor tight. No trough floor has been used on the work.



Cross Section at End of Bridge. Longitudinal Section at Center Line of Bridge.

FIG. 7. SOLID FLOOR BRIDGE WITH TIMBER DECK; TRACK ELEVATION WORK OF THE CHICAGO & ALTON RY. AND ATCHISON TOPEKA & SANTA FE RY., AT CHICAGO.
G. W. Vaughn, M. Am. Soc. C. E., Engineer in Charge

It is estimated that the life of a pine plank, the cells of which are completely filled with creosote or dead oil, will be 30 years at least, which is practically the life of a steel plate floor. For comparison of cost, Mr. Vaughn states that the steel plate floor may be taken as costing 54 cts. per sq. ft., as against 23 cts. per sq. ft. for a creosoted plank floor, assuming of course that both are to carry ballast. Mr. H. F. Baldwin, M. Am. Soc. C. E., Chief Engineer of the Chicago & Alton Ry., states that the reasons for adopting this type of floor construction were as follows: (1) The cost is lower than for an all-steel floor; (2) The steel work is all open for inspection and painting, except the tops of the I-beams; (3) The noise caused by trains is deadened as much as by any other form of construction.

ATCHISON, TOPEKA & SANTA FE RY.

On this road the only protective covering used so far on steel bridges having ballasted floors, is the ordinary brown mineral paint. The type of I-beam ballasted floor used in the bridges on the Chicago joint track elevation work has been described above. This road has not used the so-called trough floors nor asphaltic concrete filling, but has used a timber deck in preference to a steel deck. All the solid floor bridges thus far built have been constructed with transverse steel I-beams spaced from 14 to 20 ins. c. to c., and these beams are covered by a course of creosoted timber which supports the ballast. By this design a bridge is obtained in which no part of the metal work is exposed to the ballast, and all parts of the metal work can be easily got at for cleaning and painting. A number of deck girder bridges for the lines of the Eastern Ry. of New Mexico have been designed with girders spaced 10 ft. c. to c., and with a solid timber deck of creosoted timber placed over the tops of the girders. This timber deck supports a bed of ballast and the ordinary track. None of these bridges have, however, yet been erected.

DELAWARE, LACKAWANNA & WESTERN RY.

With ballasted floor construction on this road, the metal work is not covered up if it can be avoided, but is left accessible for inspection and painting. There is some track elevation work at various points where the ordinary box trough floor

is used. Mr. Lincoln Bush, Chief Engineer, informs us that in such cases he has filled the troughs with concrete, using fine material and making the mixture very wet, so as to get as solid a mass as possible. This concrete covers the entire metal work of the floor, and on top of it is laid an asphalt waterproofing course. There are several old bridges on this railway in which the box troughs have been used and the floor treated with asphalt pitch, but in such cases it is found that the waterproofing paint has not prevented the metal from rusting. Mr. Bush states that he is a firm believer in concrete for the protection of metal work.

At many of the street crossings on track elevation work, a solid wooden floor is used consisting

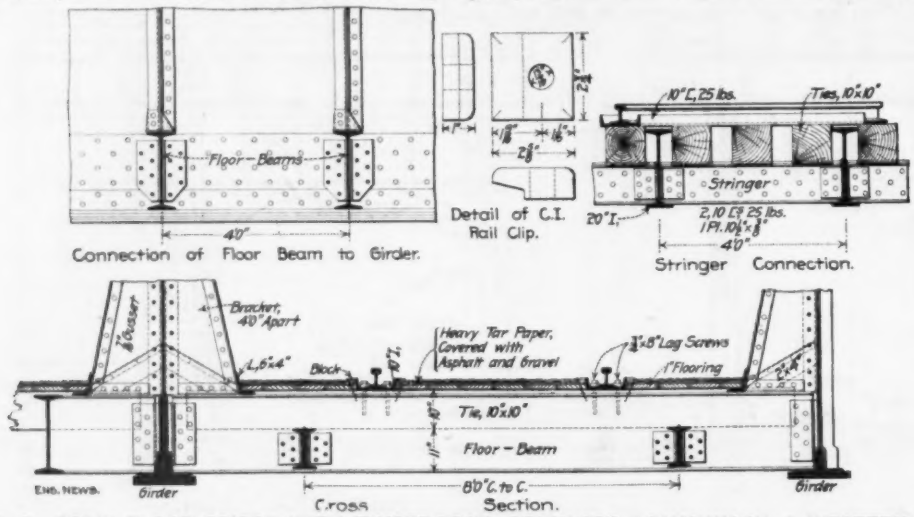


FIG. 8. SOLID FLOOR BRIDGE WITH TIMBER DECK; DELAWARE, LACKAWANNA & WESTERN RY. L. Bush, Chief Engineer.

of a matched flooring or deck, on top of which is a covering of tarred paper and gravel, as in roof construction. Some of these floors have been in service for three years, and they are found to be very quiet. They have also proved to be more nearly water-tight than any other type of floor that has been used, excepting only the trough floor with concrete filling covered with asphalt. In Fig. 8 is shown the construction of this wooden deck floor. Between the bottom chords of the main plate girders are transverse 20-in. I-beams, 4 ft. apart. On the lower flanges of these rest two stringers, 8 ft. apart, each composed of two 10-in. channels and a plate ⅜ × 10½ ins. Between each pair of I-beams are three cross-ties 10 × 10 ins., slotted to fit over the ½-in. projection of the plate in each stringer. Upon these ties are laid two 10-in. channels, forming continuous bearings for the rails, which are secured by clamps or heavy lag screws screwed into the ties. These screws are ¾ × 8 ins. A floor of 1-in. planking is spiked to the ties, and carried up on an incline between the gusset plate stiffeners of the main girders. On this is a layer of heavy tarred paper, covered with a roofing composition of asphalt and gravel.

CHICAGO, MILWAUKEE & ST. PAUL RY.

Mr. C. F. Loweth, M. Am. Soc. C. E., Engineer and Superintendent of Bridges and Buildings, states that so far this road has used nothing but paint on the track elevation solid-floor bridges. Several different kinds of paint have been used. The decks have been covered with a thin layer of gravel; this, however, was not as a preservative, but to deaden the noise. Mr. Loweth has used very few ballasted floor bridges, and these have been painted very carefully with the best paint obtainable. A lack of confidence in the durability of the metal work in such bridges has made him avoid them as far as possible, besides which for small structures he is using a form of square flat-top culvert built of I-beams and concrete, which he considers a much better structure than the ballasted floor metal bridge. For the new track elevation bridges in Chicago something better than paint is being considered, but nothing has yet been adopted. It is stated that there is so much pedestrian travel over them that an asphalt coating would have to be quite thick and have a good wearing surface to be durable.

THE USE OF THREE-POSITION, SEPARATE HOME AND DISTANT AND OVERLAPPING AUTOMATIC BLOCK SIGNALS.*

The principal matter on which this committee has to report is evidently the relative merits of the three-position and the separate distant signal as a means of giving the distant indication in connection with automatic block signals.

The overlap system is a sacrifice of principle for economy, or it presupposes that the signals will not be rigidly observed. It involves the vicious inconsistency of requiring the engineman to stop at each signal that is found in the stop position, while it is freely recognized that, under certain conditions which will arise in the proper running of a train, the stop will be impossible. The overlap indeed insures that if a stop is made as quickly as possible when a signal is overrun, no harm will result. But such incomplete signaling can hardly fail to diminish the respect of many enginemen for other stop signals and must be a severe trial to the conscientious man who wishes to obey the rules and also make time in thick weather. Whatever service the overlap system has rendered in the early development of automatic signaling, the committee feels that a point has now been reached where the saving of cost cannot justify the sacrifice of principle. Inconsistency and lack of uniformity in signaling tend to bring about disregard of signals; the constant raising of the standard of safety at the present day demands expenditure to remove such inconsistencies. The committee therefore recommends that the future installation of overlapping automatic block signals be condemned as contrary to good practice.

Whether the distant indication for one home signal should be given on the same post with the next home signal in the rear or on a separate post within the block depends, in the opinion of the committee, on the length of block. It will be assumed that the ideal distance between a distant signal and its home is practically that in which any train that is likely to run on that piece of road can be stopped. If the distance is very much greater than this, three disadvantages result. First, the engineman has to carry the distant indication in mind an unduly long time and may become confused in his recollection of it. Second, the capacity of the road for the movement of successive trains at speed is decreased. For it is obvious that trains cannot, with safety, be run habitually nearer together than the length of a block plus the distance between a home signal and its distant. Since the capacity of the road decreases as the length of block increases, there is a double loss if the distant is kept on the same post with the home when the length of block much exceeds the necessary distance for the distant signal. Third, slight delays, especially in thick weather, will become more numerous; for the farther a distant signal is from its home, the oftener it will happen that a train will reach the distant before the home is clear. In the opinion of the committee, these disadvantages outweigh the obvious advantages of concentrating the home and distant indications on one post, and it is recommended that, where the usual length of block must be much greater than one mile, the distant indication be given on a separate post.

When distant signals were first used in connection with automatic blocks of moderate length, nothing was more natural than that a separate signal should be placed on the same post with and below the home; this was simply following the practice that had already been established both in England and America in connection with mechanically operated signals. The use of a single three-indication signal to do what had been done by the two signals, was a later conception. This was first used on the Pennsylvania Lines West of Pittsburg in the summer of 1899 and was in regular service the following year. Since that time it has been extensively installed on those lines and is standard. These signals are also used on the Pere Marquette.

The arguments offered in favor of the three-position signal against the separate home and distant are:

1. There being but one signal arm or one light to watch, the indications are simpler and more easily understood by the engineman.

2. The indication is the same as that given by the station or telegraph block signals; the three-position signal conforms with the interlocked signals more nearly than the separate home and distant, and in every way conforms to correct principles.

3. The cost of installation is less; the three-position (electric semaphore) signal costs $210, while the two-arm signal costs $300.

4. The three-position signal of the improved type, having nearly one-half fewer parts, is less complicated than the two-arm signal.

5. The cost of maintenance is less; there is but one lamp to keep up and, as there is but one pair of lock or clutch magnets, the current consumption is reduced almost one-half.

It is stated that, in a certain territory, a number of train crews have part of their run on a road equipped with three-position automatic signals and part on tracks having automatic signals with home and distant on the same post; several of the enginemen were questioned as to their preference and all answered that they liked the three-position signals better.

On the side of the separate home and distant arrangement it is claimed:

1. The vertical clear position of the blade of the three-position signal is objectionable because it is not sufficiently conspicuous and distinctive; the blade would take this position in case of a breakage of the semaphore casting between the bearing and the up-and-down rod; and, it is believed, if the blade is broken off, its absence might be overlooked and be taken for a clear signal.

2. In comparison with the separate home and distant on the same post, the three-position signal has only one light, and when one light is out all are out; there is then no marker showing the location of the signal, and it may easily be overrun. This is not true of the other system, as the chance that two lights on the same post will be out at the same time is very remote. Where two lights are used, always in the same relative position, the indication becomes more distinctive and less likely to be confused with other lights.

3. Again comparing the three-position signal with the home and distant on the same post: Indications for two blocks are given at each semaphore. The mental process of reading two separate indications is simpler than getting the information from one light or one blade—the upper blade, or upper light, refers to the first block and to nothing else, and the lower blade, or light, to the second block and nothing else. The distant signal blade (by day) exactly repeats the positions of its home, while a certain position of the three-position signal represents an entirely different position of the signal in advance.

4. Consistency and uniformity are sacrificed, not enhanced, by using the same signal for automatic block work as for telegraph block and making the caution and clear positions of the signal serve as distant indications for the next signal ahead. At a telegraph block station the caution indication of the signal means, "There is a train in the block ahead; proceed with caution;" as an automatic block signal it means, "There is no train in the block ahead, but the next signal is at danger." The clear indication of the telegraph block signal means merely that the block ahead is clear, but gives no clue to the indication of the next signal; as an automatic signal, it means that the block ahead is clear and also that the next signal is not in the stop position. These are radical differences between the significations of the same signals when found at different places along the same road.

The committee thanks a large number of members who sent replies to its circulars, several of which were very complete discussions of the subject. Ninety-five per cent. of these replies favor separate home and distant signals, and, of course, the greater part of the automatic block signals now in service are of that type. The question is, therefore, not merely whether the three-position system is preferable to the other, but whether it is so far superior as to justify an effort to make it a standard for general adoption at the cost of ultimately changing the separate home and distant signals now in use. Good arguments are advanced on both sides, but the unanimous conclusion of the committee is that the two are pretty evenly balanced. The committee, therefore, finds no sufficient reasons for recommending that the common practice of using separate home and distant signals on the same post should be departed from in favor of the three-position signals.

C. C. Anthony, G. B. Gray, W. W. Slater, E. D. Wileman, A. R. Raymer, Committee.

*Report of a Committee at the Annual Meeting of the Railway Signaling Club.

CIVIL SERVICE EXAMINATION FOR ENGINEERS ON THE NEW YORK BARGE CANAL.

In view of the fact that the referendum vote for the proposed $101,000,000 barge canal in New York state has resulted in a substantial majority in favor of the canal, it is likely that work will be actually begun within six months. According to newspaper reports, the State Engineer and Surveyor will advertise the first contracts for locks and rock excavation so that work may begin by May 1, 1904, and the canal work will continue about seven years. This means that a large number of engineers, levelers, inspectors, rodmen, chainmen and laborers will be appointed by the State Engineer.

In answer to a letter of inquiry addressed to the State Civil Service Commission at Albany, N. Y., we have received the following reply:

Sir: Replying to yours of Oct. 5, I have to say that arrangements have not been made for examinations for engineering positions on the barge canal, but it is probable that such examinations will be held in December or January.

I send you under separate cover our manual of examinations, which outlines the requirements for engineering positions in the state service and gives specimen questions of the principal examinations (see pages 41-47). You are at liberty to publish any of the matter found in the manual which you think would be of interest to your readers.

An examination for the position of engineering draftsman to fill vacancies now existing in the office of the State Engineer and Surveyor will be held Nov. 28.

Yours very respectfully, Chas. S. Fowler,
Albany, N. Y., Nov. 6, 1903. Chief Examiner.

From the "Manual of Examinations" we have abstracted the following relative to requirements for the various positions on the State Engineering Corps:

ASSISTANT CIVIL ENGINEER.—Salary $5 to $6 per day when employed. Open to men only. Candidates must have had at least three years' practical experience, one year of which must have been in responsible charge of work. Candidates who have pursued successfully for three years or more a course of study in civil engineering in a school maintaining a standard satisfactory to the examining board will, upon presenting proper evidence of that fact, be credited with one year of the required experience. Time allowed, eight hours.

Subjects of examination.	Relative weights.
1. Theoretical and practical questions, including highway construction, strength of materials, engineering construction, hydraulics, mechanics of engineering, specifications and estimates, etc.	10
2. Experience	7
3. Education	3
Total	20

LEVELER.—Open to men only. Salary $4.50 to $5 per day when employed. Candidates must have had at least two years' practical experience. Candidates who have pursued successfully for three years or more a course of study in civil engineering in a school maintaining a standard satisfactory to the examining board will, upon presenting proper evidence of that fact, be credited with one year of the required experience. Time allowed, eight hours.

Subjects of examination.	Relative weights.
1. Theoretical and practical questions, including mensuration and use of logarithms, plane trigonometry, topographical surveying and leveling, elementary mechanics, theory and use of rod, level and transit, highway construction, etc.	5
2. Experience	3
3. Education	2
Total	10

RODMAN.—Salary $3.50 to $4 per day when employed. Open to men only. No practical experience is required. Time allowed, six hours.

Subjects of examination.	Relative weights.
1. Theoretical and practical questions, including mensuration and use of logarithms, plane trigonometry, elementary surveying and leveling, theory and use of rod, level and transit, highway construction	6
2. Experience	1
3. Education	3
Total	10

CHAINMEN.—Minimum age, eighteen years. Open to men only. Salary $2.50 to $3 per day when employed. Time allowed, six hours. No practical experience is required. Subjects of examination and relative weights: Arithmetic and use of chain, 8; experience and education, 2.

ENGINEERING DRAFTSMAN.—Open to both sexes. Salaries $4 to $5 per day. The examination will relate entirely to the duties of the position and will consist, in part, of practical drafting. Time allowed, eight hours.

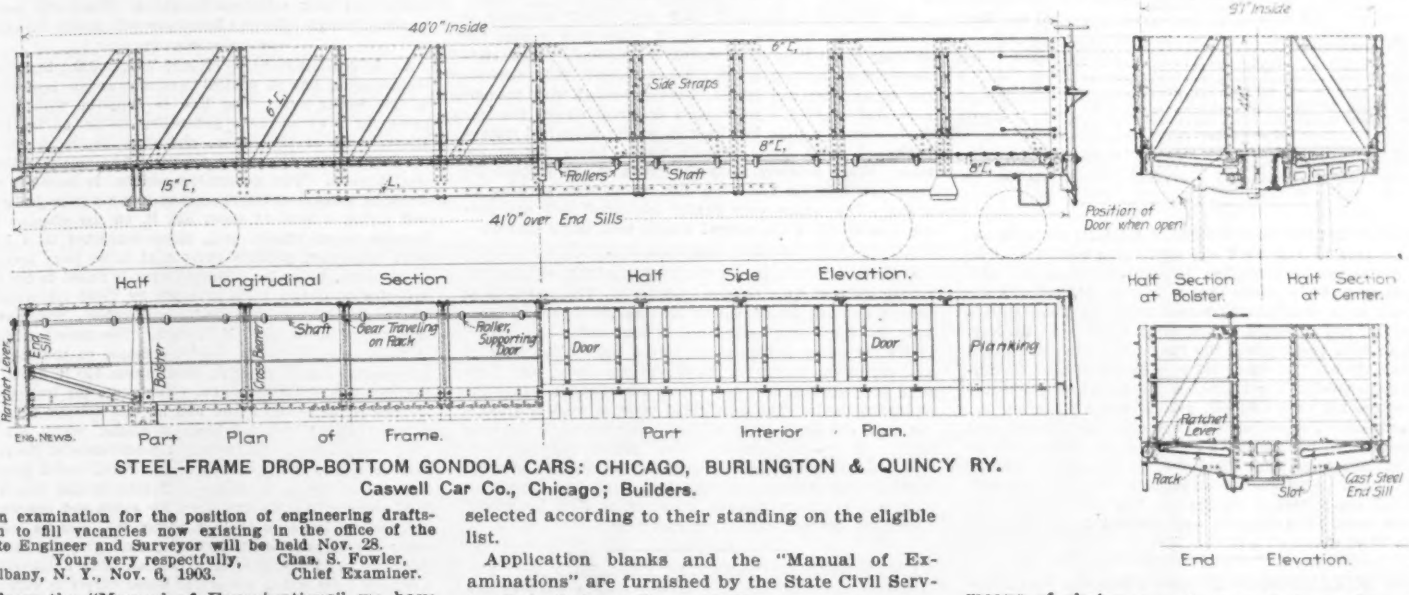
INSPECTOR OF PUBLIC WORKS.—Salary $5 per day when employed. Open to men only. Time allowed, six hours.

Subjects of examination.	Relative weights.
1. Education and experience	4
2. Practical questions on materials, construction and inspection	6
Total	10

It should be stated that under the law governing the barge canal work, the inspectors are members of the State Engineering force. Previous to this the inspectors have been appointed by the Superintendent of Public Works, and were not members of the corps of engineers. This change should prove a good one for all concerned.

We may remark in closing that no political pull is necessary to secure an appointment, men being selected according to their standing on the eligible list.

Application blanks and the "Manual of Examinations" are furnished by the State Civil Service Commission, Albany, N. Y.



STEEL-FRAME DROP-BOTTOM GONDOLA CARS: CHICAGO, BURLINGTON & QUINCY RY.
Caswell Car Co., Chicago; Builders.

DROP-BOTTOM, FLAT-FLOOR GONDOLA CARS FOR THE CHICAGO, BURLINGTON & QUINCY RY.

While a number of types of drop-bottom and self-clearing cars are in service, most of them cannot be conveniently employed in general freight service or when the load has to be shoveled out. The Chicago, Burlington & Quincy Ry. is about to put in service 2,000 steel-framed gondola cars of 100,000 lbs. capacity, which are of a drop-bottom, level-floor type, adapted for unloading either by dumping or shoveling, and suitable for carrying coal, ore, gravel, etc., as well as for ballasting or filling. These cars are of a type originally patented by Mr. W. A. Caswell, and described in our issue of Aug. 31, 1899, but a number of improvements have been introduced in the new cars. Of the 2,000 cars, one lot of 1,000 will be of steel construction throughout, with 15-in. channels for the center sills. The sides are of plate-girder construction, having ¼-in. web plates, angle iron stiffeners, an outside flange angle at the top, and an inside flange angle at the bottom for attachment to the floor, which is also of ¼-in. plate. The door openings are 38⅝ × 38 ins., and the doors are operated by chains wound upon longitudinal shafts under the sides of the cars. Another method of operating the doors is preferred by the car company, and is described below, but the railway company will try both methods, each on 1,000 cars. The other lot of 1,000 cars will be of combination construction, with steel framing and wooden sides, ends and floor.

All the cars are built under the patents of the Caswell Car Co., 1610 Fisher Building, Chicago. The 1,000 steel cars are being built by the Cambria Steel Co., of Johnstown, Pa., while the 1,000 combination cars are being built by the Standard Steel Car Co., of Pittsburg, Pa.

The accompanying cut shows the construction and operating mechanism of the combination cars, but the floor arrangement is the same in all. The special feature of the Caswell cars is in the floor construction, and the floor is composed mainly of 16 trap doors, hinged at the inner side. When these doors are dropped there is no side obstruction to the flow of the contents of the car, and only about 5% of the contents remains on the spaces between the doors. This material is removed by shoveling, and at stations where there are no facilities for dumping the car can be entirely unloaded by shoveling, in the usual way. The doors are omitted from the corner panels in order to give the greater strength due to solid construction at these points. The doors are operated in four groups of four each, so that portions of the load can be dumped if required. The doors can also be held at any position, and in this way the flow of ballast or other material can be regulated.

In the original design the doors were raised by means of chains wound upon shafts under each side of the car, and were locked in position by catches. This plan, as above noted, is being used on 1,000 of the new cars. The other cars, shown in the accompanying cut, are fitted with an ingenious door-operating mechanism which dispenses entirely with the use of chains. This latter method is preferred by the Caswell Car Co., as it is thought that unequal stretching of so many chains may make it difficult to close the doors properly. On the combination cars, shown herewith, there are four shafts, one to each set of doors, and on each shaft are journaled two rollers under each door, the rollers bearing against iron straps. The shafts pass through slots in the end sills, bolsters and cross bearers, and each of these slots has a rack on its lower surface. Keyed on each shaft are pinions engaging with the racks, the pinions being shrouded to prevent displacement. The outer end of the slot is level, while the rest of the length is inclined downward. With the doors closed, the shaft pinions rest on the level part of the rack and support the direct vertical load of the door, the rollers holding the doors up against a triangular timber bolted to the inner face of the side sill. To open the set of doors, the shaft is turned by a ratchet handle at the end of the car. The gears then travel down the racks and carry the shaft toward the center of the car, letting the doors gradually downward. It will be seen that the doors are supported at every position, while longitudinal rods prevent the doors from dropping far enough to strike the wheels or trucks.

It will be seen that in the combination cars, the center sills are 15-in. steel channels, set back to back, about 12 ins. apart, and covered by a ¼-in. steel plate riveted to the flanges. These

are reinforced near the middle of the car, by angles riveted to the lower edges. The end sills and bolsters are steel castings, while the cross bearers at intermediate panel points are steel channels. The side sills are 8-in. channels, set with their lower flanges level with the top flanges of the center sills, but at each corner an extra sill of 8-in. steel channel extends back from the end sill to the bolster. There is also a diagonal brace at each corner, as shown in the plan. By thus raising the side sills, they offer no obstruction when the load is being dumped. The side posts are of T-section, formed by two angles riveted together, while the end posts and corner posts are single angles. The diagonals are 8-in. channels, riveted to the 8-in. side sills and a 6-in. top channel. The sides and ends are of plank, secured by bolts passing through the posts, with straps on the outside. The central part of the floor and the corner sections are covered with planking.

The cars are mounted on the standard freight car trucks of the Chicago, Burlington & Quincy Ry., which are of the arch-bar or diamond type, with 33-in. wheels, but having bolsters set low to accommodate the drop doors. The general dimensions of the cars are as follows:

Length over end sills	41 ft. 0 ins.
Length inside	40 " 0 "
Width over sills	9 " 6 "
Width inside	9 " 1 "
Depth inside	4 " 4 "
Height, rail to top of car	8 " 4⅜"
Wheelbase, truck	5 " 2 "
Wheelbase, total	34 " 4½"
Distance c. to c. of trucks	29 " 2¾"
Cubic capacity	55 cu. yds.
Carrying capacity	100,000 lbs.
Weight, empty	39,500 "

BIDS FOR THE BLACKWELL'S ISLAND BRIDGE superstructure at New York city were again opened on Nov. 5 by Bridge Commissioner Lindenthal. Bids for this structure were previously opened on Sept. 24, but only one bid was then received, the bidder being the Pennsylvania Steel Co., and the price named being $5,255,514. On Nov. 5 there were two bidders, the Pennsylvania Steel Co. and Milliken Bros. The prices bid were $5,132,985 and $5,188,850, respectively. For 12,200,500 lbs. nickel steel eye-bars the prices bid were 8.03 cts. per lb. and 8.1 cts. per lb., respectively. For carbon steel eye-bars the prices named were 6.62 cts. per lb. and 5.9 cts. per lb. For 69,550,000 lbs. of structural steel the prices were 5.64 cts. per lb. and 5.7 cts. All these prices include erection. The bids in detail are printed in the Construction News Supplement in this issue. The contract was awarded to the Pennsylvania Steel Co.

THE WILLIAMSBURG BRIDGE across the East River is now so near completion that it is expected to have it formally opened to vehicle and pedestrian traffic on Dec. 1. The question of franchises for electric surface car lines across the bridge has not yet been settled.

THE MANHATTAN TERMINAL OF THE BROOKLYN bridge is to have four additional loops for surface cars in service on Nov. 16. The construction of these loops was recommended nearly two years ago by a commission, made up of Messrs. Gustav Lindenthal, Wm. Barclay Parsons and Geo. B. Post, as the best means of relieving the crowding at the terminal.

HEAVY TERMINAL TRAFFIC is shown by recent figures of the train service at the South Station, Boston, Mass. The station was opened early in 1899, and the traffic of all tributary lines had been diverted to it by the middle of that year. The increase since then is shown in the following table:

Number of Regular Trains Passing In and Out of South Terminal Station, Boston, Mass.

Date.	No. of trains.
Oct. 22, 1899	714
Oct. 21, 1900	740
Nov. 3, 1901	758
Oct. 19, 1902	787
Oct. 18, 1903	840

This figure is probably the world's record for traffic in a terminal station.

MORE NAVY YARDS AND DRY DOCKS are needed by the U. S. Navy, according to the annual report of Rear-Admiral Endicott, Chief of the Bureau of Yards and Docks of the U. S. Navy. New large dry docks are urgently needed at the Pensacola (Fla.) and the Puget Sound Navy Yards, and smaller ones, for torpedo boats, etc., should be built at the Norfolk (Va.) and other navy yards. The principal items of work which continue through the coming year are work on the dry docks at Norfolk (Va.), Charleston (S. C.), and New York (N. Y.); and the floating dock for Cavite (P. I.).

THE PROPERTY OF THE MAINE WATER CO., or rather of the portion of it located in Waterville, Fairfield, Winslow and Benton, Me., has been appraised at the instance of the Kennebec Water District at $503,476, including the value of the "plant, property and franchises." These waterworks supplied Waterville and Fairfield and are being acquired under the water district act of 1899, outlined and discussed editorially in our issue of May 28, 1903. As stated in that issue, the constitutionality of the act creating this water district was sustained by the highest courts of Maine. The appraisers, appointed by the Supreme Court, were Judge Frederick A. Powers, Mr. Frederick H. Appleton, and Mr. Percy M. Blake, the latter being a well-known Boston engineer. The case was warmly contested and a number of experts were employed by each side. Among the experts on behalf of the District were F. C. Coffin, John R. Freeman, Wm. R. Hill, J. W. Alvord, F. C. Whipple and Leonard Metcalf. Among the experts on behalf of the company were Charles A. Allen, Wm. Wheeler, Prof. Caird, W. B. Getchell and Robert Milligan. All but the last three are members of the American Society of Civil Engineers. The hearings began on Oct. 6, 1903, and the award is dated Nov. 2, the valuation to be as of Nov. 1.

THE MOST SERIOUS RAILWAY ACCIDENT of the week was the derailment of a freight train on the Cleveland, Cincinnati, Chicago & St. Louis Ry., near Marion, O., on Nov. 4. Two of the trainmen were killed.

AN UNUSUAL STREET CAR ACCIDENT occurred in Kansas City, Mo., on Nov. 4. A cable line crosses the railway yards over the Twelfth St. viaduct, which rises eastward on a steep grade. An eastbound train, consisting of grip-car and trailer, heavily loaded with passengers, had reached a point some distance beyond the viaduct, when the train lost its grip on the cable and started backward down grade. There was a heavy fog at the time. When halfway down the grade of the viaduct the train ran into the following one, which was working up grade. The trains were partly telescoped and one person was killed, while several others were badly injured.

A U. S. NAVAL MAGAZINE EXPLODED on Nov. 4, killing six men who were at work in it. The magazine is located on Iona Island, in the Hudson River, near Peekskill, N. Y. Large quantities of explosives and loaded shells for naval ordnance are stored here. The bulk of the explosive matter is kept underground, but there were four shell-houses containing considerable quantities of loaded shells. The men killed were at work overhauling some old shells when a 13-in. shell exploded; the contents of two shell-houses were ignited and exploded. The underground magazines were fortunately not affected. It is recalled that an explosion from a similar cause, but with less extensive damage, occurred at Fort Lafayette, New York harbor, during the past year.

A CAR LOAD OF DYNAMITE EXPLODED Nov. 1 at Crestline, O. The car was in the freight yard of the Pennsylvania R. R., and the dynamite is supposed to have been exploded by a jar during switching. Some 500 freight cars were set fire by the explosion, and hundreds of people received minor injuries from the shock and by being struck with flying glass. Locomotives a mile away are said to have been derailed. Two men half a mile away were seriously and perhaps fatally injured by the destruction of a building. A crater 40 ft. deep is reported to have been blown in the ground where the car loaded with dynamite stood.

QUICKSAND HAS CAUSED TRACK SETTLEMENT on the Pennsylvania and Lake Shore Railroads at Cleveland, O., to such a degree that the loss already amounts to $250,000, according to newspaper reports. The walls of the new Pennsylvania freight house have collapsed, injuring five men, two fatally.

THE GUAYAQUIL & QUITO RY. has stopped construction work, owing to the refusal of the Government of Ecuador to issue bonds for further work. The road has reached Guamote. One stretch of 60 miles has cost $16,000,000.

A CONFERENCE OF EASTERN HYDROGRAPHERS, called by Mr. F. H. Newell, Chief Engineer of the Hydrographic Division of the Geological Survey, was held in Washington, Oct. 28 to 31. The following districts and divisions of the work were represented: New England, Mr. N. C. Grover; New York, Mr. Robert E. Horton; Central States, Mr. E. G. Paul; Southern States, Mr. M. R. Hall; Mississippi Valley States, Mr. E. Johnson, Jr.; General Inspection, Mr. E. C. Murphy; Washington office, Messrs. G. B. Hollister and John C. Hoyt; Hydro-economics, Mr. M. O. Leighton; Hydrology, Mr. M. L. Fuller. At the opening of the session, Director of the Survey Charles W. Walcott and Chief Engineer F. H. Newell gave brief addresses. In the general discussions that followed, the principal details of the work were carefully considered. Considerable time was spent in considering the nature of the demands for hydrographic data, as made both by the engineering and the general public. Ways and means whereby these demands may best be met were fully discussed, and a special effort will be made to distribute the gaging stations and to adjust the methods and reports so as to best provide for the wants of the largest number. The determination of the profiles of rivers, which work has just been started, was also discussed. It is planned to extend the surveys so as to cover the more important rivers. Plans were made for the preparation of a manual on hydrographic methods. In this publication it is proposed to bring together the best hydrographic methods, as developed by the hydrographers of the U. S. Geological Survey and by other engineers. There has been a large call for this publication, both from the Survey's corps of engineers and from engineers in private work.

THE SIMPLON TUNNEL main headings have reached an aggregate depth of about 17,500 m. (57,400 ft.), of which 7,275 m. is in the south end, and 9,950 m. on the north end. As the total length of the tunnel will be 19,729 m., there remains a distance of about 2,200 m. still to be pierced. The aggregate advance of the two main headings during the month of September was 309 m. (1,013.5 ft.) so that about seven months' work will see the headings through. The geological conditions have been fairly favorable for a year or more past, though occasional soft seams or insecure rock require timbering for short distances. The material encountered during September in the south heading was crystalline limestone, then micaschist and later schistose limestone. The north heading passed through schistose limestone with pyritic inclosures.

A LONG-DISTANCE HIGH-VOLTAGE ELECTRIC transmission line in central Mexico was put into service a few weeks ago. The line is supplied with electric power by a water-power generating station on the Duero River, state of Michoacan, and runs thence to the city of Guanajuato, 110 miles distant. The line-potential is 60,000 volts. The generating station is supplied with water by a canal and pipe, aggregating six miles in length, and giving a head of about 325 ft. on the wheels. Two impulse water-wheels, each direct-connected to a 1,500-KW. generator, produce current at 2,300 volts pressure, three-phase, 60 cycles. This current is raised to the line-pressure of 60,000 volts pressure by three transformers. These are oil-insulated and water-cooled, and have a capacity of about 1,000 KW. each. The three conductors of the transmission line are bare copper on brown-glazed insulators, carried on 40-ft. steel towers 440 ft. apart. At the receiving substation near Guanajuato the line-current is reduced to 15,000 volts, and at this voltage it is distributed to various neighboring mines, which are the chief consumers of the power. Transformers at the mines further reduce the potential to 460 volts, which pressure is used directly on the motors. Current is also sold to the local electric lighting company, for street and commercial lighting; it is expected that power will also be sold to other towns, ranches, flour mills, etc. The line and the fixed works of the generating station have a capacity of 8,000 HP., for which power the water supply is sufficient. The generating machinery now installed has about half this capacity. The plant is owned and operated by the Guanajuato Electric Light & Power Co., a corporation of Americans. The president of the company is Mr. H. Hine, of Colorado Springs, Colo.; Mr. H. H. Filley is Chief Engineer, and Mr. R. F. Doble, of San Francisco, is Consulting Engineer.

WORKING NIGHT SHIFTS IS SELDOM ECONOMICAL and this fact is well brought out in the following statement of Robert Finch, Supt. American Bridge Co. at its Toledo plant, as given in the "Iron Trade Review":

Notwithstanding the fact that we have taken off the night force at the plant of the American Bridge Co., we have increased the production of the plant 10% by doing so. When we took off the night force, we figured that we would do just as much work, but we have been surprised to find that we are doing 10% more with our reduced force. Ever since we have been in our new plant we have had trouble keeping men at work on our night shift. They would only stay on at nights until they could get day work some place and the result was that we were breaking in new men all the time and reorganizing our force once and twice a month. Later we tried the scheme of shifting the force, giving them two weeks days and two weeks nights, but this worked no better. Then we decided to cut off the night force altogether, letting out about 100 men, and the result has been pleasing, indeed. We can keep the men long enough so that they can get acquainted with their work and their surroundings and besides getting more work we are having fewer accidents.

THE HISTORICAL LOCOMOTIVE "NOVELTY," built by Braithwaite and Ericson to compete with Stephenson's "Rocket" and Hackworth's "Sans Pareil" in the famous trials at Rainhill (England) in 1830, has been discovered recently. The "Rocket" won the prize of $2,500, as being the most practicable machine, making a speed of 29 miles an hour. The "Sans Pareil" attained a speed of only 23 miles an hour, while the "Novelty" had only run a few miles when defects in its boiler put it out of the competition.